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NATIONAL DAM INSPECTION PROGRAM. ELMHURST DAM (NDS ID 296), SUS--ETC(U)
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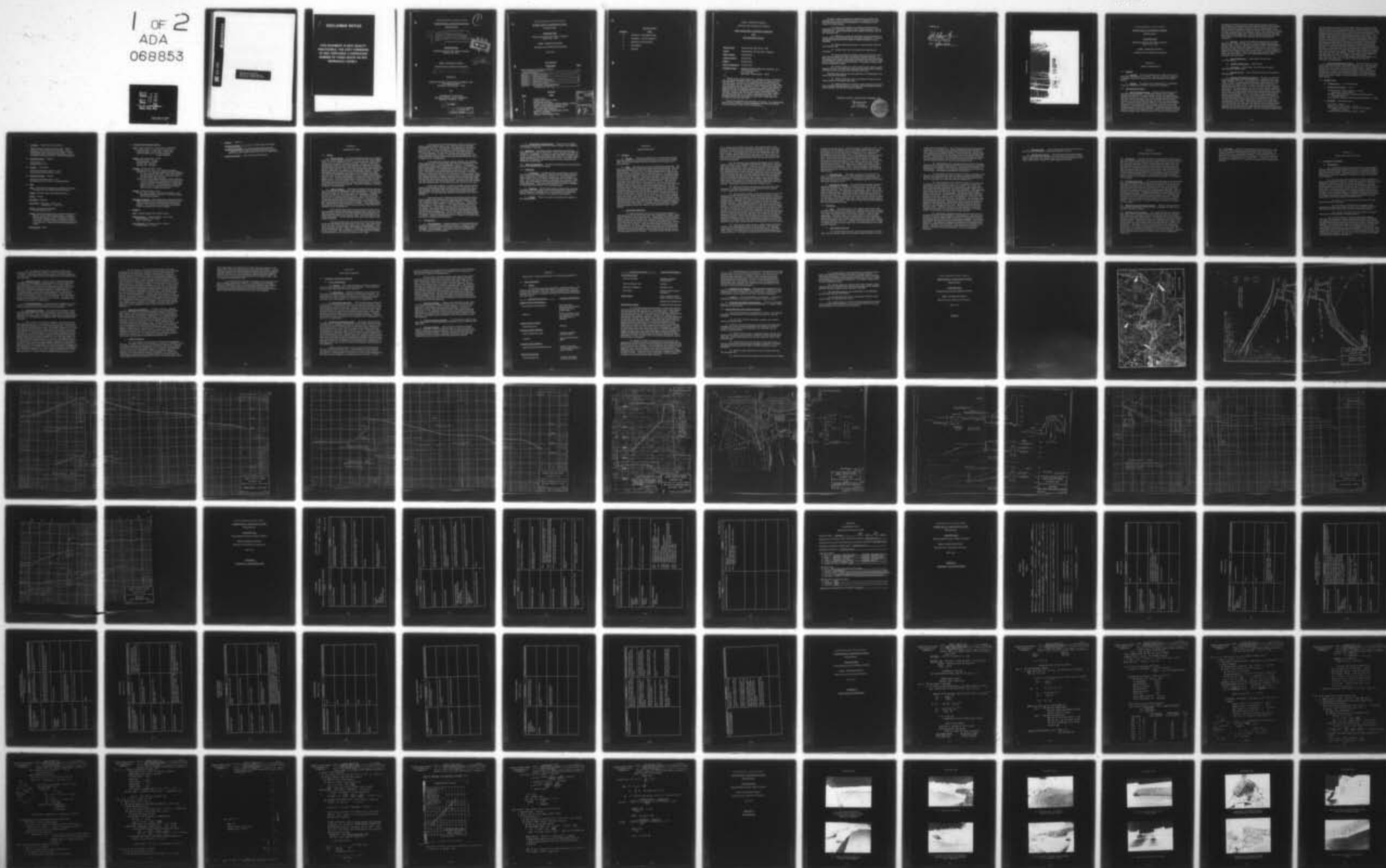

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SUSQUEHANNA RIVER BASIN

ROARING BROOK, LACKAWANNA COUNTY

PENNSYLVANIA

6 National Dam Inspection Program. Elmhurst Dam (NDS ID 296), Susquehanna River Basin, Roaring Brook, Lackawanna County, Pennsylvania. Phase I Inspection Report.

ELMHURST DAM

PENNSYLVANIA GAS AND WATER COMPANY
(NDS ID No. 296)

12 100p.

PHASE I INSPECTION REPORT

NATIONAL DAM INSPECTION PROGRAM

15 DACW31-78-C-0046

Prepared by

GANNETT FLEMING CORDDRY AND CARPENTER, INC.
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Harrisburg, Pennsylvania 17105

For

DEPARTMENT OF THE ARMY
Baltimore District, Corps of Engineers
Baltimore, Maryland 21203

11 MAY 1978

SUSQUEHANNA RIVER BASIN

ROARING BROOK, LACKAWANNA COUNTY

PENNSYLVANIA

ELMHURST DAM

PENNSYLVANIA GAS AND WATER COMPANY
(NDS ID No. 296)

PHASE I INSPECTION REPORT

NATIONAL DAM INSPECTION PROGRAM

MAY 1978

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PHASE I INSPECTION REPORT

NATIONAL DAM INSPECTION PROGRAM

BRIEF ASSESSMENT OF GENERAL CONDITION AND RECOMMENDED ACTION

Name of Dam:	Elmhurst Dam (NDS ID No. 296)
Owner:	Pennsylvania Gas and Water Company
State Located:	Pennsylvania
County Located:	Lackawanna
Stream:	Roaring Brook
Date of Inspection:	10 April 1978
Inspection Team:	Gannett Fleming Corddry and Carpenter, Inc. Consulting Engineers P.O. Box 1963 Harrisburg, Pennsylvania 17105

ABSTRACT

Based on the visual inspection, available records, calculations and past operational performance, Elmhurst Dam is judged to be in good condition. However, the spillway (stone masonry gravity and concrete chute) will not pass the Probable Maximum Flood (PMF) without overtopping the dam. Therefore, based on criteria established for these studies by the Department of the Army, Office of the Chief of Engineers (OCE), the spillway capacity is rated as inadequate. The spillway will pass one-half the PMF without overtopping the dam. Therefore, based on the OCE criteria established for these studies, the spillway capacity is not rated as seriously inadequate. The existing spillway can accommodate a flood with a peak inflow of 80 percent of the PMF peak flow.

Due to the potential for overtopping of the dam, it is recommended that the Owner develop a detailed emergency operation and warning system for Elmhurst Dam as soon as practical.

ABSTRACT

In order to correct operational, maintenance and repair deficiencies and to more accurately determine the condition of the dam, the following measures are recommended to be undertaken by the Owner in a timely manner:

(1) Repair stone masonry to eliminate leakage through downstream face of gravity section of dam, gatehouse walls, and left abutment wall. Repair deteriorated stone masonry along top 3 feet of left abutment wall.

(2) Operate the valves on the gated outlets periodically to ensure they will be functional during emergency conditions. Lubricate the operating mechanisms and repair or replace hazardous access facilities.

(3) Repair concrete extensions on right abutment wall and left abutment wall.

(4) Monitor right wall of concrete chute spillway for movement.

(5) Evaluate effect of possible improper functioning of concrete chute spillway wall drain with respect to design function. If drain must be functional, install several observation wells in the drain to determine the extent of hydrostatic pressure on the wall and to determine if repairs to the drain are necessary. If installed, monitor wells to see if condition worsens.

(6) Perform repairs on surface runoff control system, outlet channel left wall, concrete apron joint below masonry gravity spillway, and hole at base of concrete chute spillway right wall.

The following measures are recommended to be undertaken by the Owner when the need arises:

(1) Provide round-the-clock surveillance of Elmhurst Dam during periods of unusually heavy rains.

(2) When warnings of a storm of major proportions are given by the National Weather Service, the Owner should activate his emergency operation and warning system procedures.

GANNETT FLEMING CORDDRY AND CARPENTER, INC.

A. C. Hooke

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Head, Dam Section

COMMONWEALTH OF PENNSYLVANIA
REGISTERED PROFESSIONAL
ALBERT CHARLES HOOKE
ENGINEER
No. 2201-E

APPROVED BY:

John H Kenworthy

JOHN H. KENWORTHY
LTC, Corps of Engineers
Acting District Engineer

DATE: 6 June 1978

ELMHURST DAM

Elmhurst Dam — Looking Upstream

SUSQUEHANNA RIVER BASIN

ROARING BROOK, LACKAWANNA COUNTY

PENNSYLVANIA

ELMHURST DAM

PENNSYLVANIA GAS AND WATER COMPANY
(NDS ID No. 296)

PHASE I INSPECTION REPORT

NATIONAL DAM INSPECTION PROGRAM

SECTION 1

PROJECT INFORMATION

1.1 General.

a. Authority. The Dam Inspection Act, Public Law 92-367, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a program of inspection of dams throughout the United States.

b. Purpose. The purpose of the inspection is to determine if the dam constitutes a hazard to human life or property.

1.2 Description of Project.

a. Dam and Appurtenances. Elmhurst Dam consists of the following primary features: an earthfill structure with a central masonry core wall and a masonry gravity section that comprise the main part of the dam, a masonry gravity structure extending upstream that forms the left abutment wall, a low earthfill structure with a masonry core wall extending upstream that forms the right abutment wall, a masonry gravity overflow spillway, a concrete chute spillway constructed on the embankment to the right of the masonry gravity spillway, an earthfilled pier separating the spillways, and a large concrete apron to serve both spillways. Control facilities are located on the left side of the masonry gravity spillway and include an intake structure located about 90 feet upstream of the dam in the reservoir,

an approach channel and tunnel connecting the intake structure to a screen chamber containing two conduit intakes located at the upstream face of the gravity section, another section of masonry tunnel, and three gatehouses located downstream of the masonry gravity section of the dam, again connected by a masonry tunnel. Two emergency conduits, a 48-inch cast-iron pipe, and a 36-inch cast-iron pipe are controlled from the two lower gatehouses. Various features of the dam are shown on the Plates at the end of the report and on the Photographs in Appendix D.

b. Location. The dam is located on Roaring Brook in Lackawanna County, Pennsylvania, about 11 miles southeast of Scranton, Pennsylvania. Elmhurst Reservoir is shown on USGS Quadrangle Sheet, Moscow, Pennsylvania, with coordinates N41°22'45" - E75°31'50". The location map is shown on Plate 1. Reservoirs located upstream of Elmhurst Dam include Curtis Dam, located about 1.4 miles upstream on White Oak Run; Hollister Dam, which presently does not impound any water; and Lake Henry, a very small impoundment located well upstream on Roaring Brook.

c. Size Classification. Intermediate (64 feet high, 3,744 acre-feet).

d. Hazard Classification. High hazard.

e. Ownership. Pennsylvania Gas and Water Company, Wilkes-Barre, Pennsylvania.

f. Purpose of Dam. Water supply for Dunmore and Scranton, Pennsylvania.

g. Design and Construction History. The dam was designed for the Scranton Gas and Water Company by E. Sherman Gould, Consulting Engineer, in 1887. Construction was completed in November 1889. In 1899, for the purpose of increasing storage capacity, the masonry gravity spillway and core wall were raised 3 feet and the embankment was raised 1 foot. In 1902, the right abutment wall was raised to the same elevation as the top of the dam and was extended accordingly. The left abutment wall was raised to within 1 foot of the top of dam. A 10-inch wall drain was also constructed along the back face of the left abutment wall. No records of design were reviewed for these modifications. In 1914, the Water Supply Commission of Pennsylvania performed a study of the principal features of Elmhurst Dam that included stability, hydrologic, and hydraulic analyses. As a result of the study, improvements for the safety of the dam were recommended to the Owner. These improvements were completed in 1916 and included the following: the masonry gravity spillway height was reduced by 3 feet, and the upper level of the masonry gravity spillway was strengthened by drilling vertical holes and grouting steel bars in them near the upstream face. The dam functioned satisfactorily

until the flood of August 1955, which was caused by Hurricane Diane and is the flood of record at Elmhurst Dam. During that flood, the left abutment wall was overtopped by 6 inches and was severely undermined. Maximum water level was within 6 inches of top-of-dam elevation. The adjacent railroad trackage was damaged and had to be re-built. The masonry apron below the masonry gravity spillway also suffered major damage. After that flood, repairs and improvements were completed in 1958 and included the following: a concrete chute spillway was constructed on the embankment adjacent to the masonry gravity spillway, the damaged masonry apron was repaired by placing a concrete slab over it and the apron was extended across the width of the concrete spillway, a deep cutoff wall was constructed at the downstream end of the apron, the left abutment wall was raised with concrete to the elevation of top of dam and a concrete apron was placed along the left abutment wall just under the backfill. Records of construction and design data are on file for these improvements and repairs. The engineering was performed by Thomas H. Wiggin, Consulting Engineer, of New York, New York. The concrete spillway was model tested at the Alden Hydraulic Laboratory, Worcester Polytechnic Institute, Worcester, Massachusetts.

h. Normal Operational Procedures. The valves for the 36-inch water supply line, located in the upper gatehouse, are normally open, and the valve in the lower gatehouse is normally open, thus allowing withdrawal of water into the water distribution system. Another 36-inch valve, which is in the lower gatehouse and would allow the 36-inch line to serve as an emergency conduit, is normally closed. The 48-inch valve located in the middle gatehouse on the emergency conduit is normally closed, but during periods of low flow, it is partially opened.

1.3 Pertinent Data.

a. Drainage Area. 37.3 square miles.

b. Discharge at Damsite. (cfs.)

Maximum known flood at damsite - 14,500 (estimated - August 1955).
Emergency drawdown lines at maximum pool elevation - 840 (approximate).
Total spillway capacity at maximum pool elevation - 31,000.

c. Elevation. (Feet above msl.)

Top of dam - 1431.5.
Maximum pool (top of dam) - 1431.5.
Normal pool (spillway crest - masonry and concrete spillways) - 1422.5.
Upstream tunnel invert outlet works - 1378.5.

c. Elevation. (Feet above msl.)(Cont'd.)

Upstream invert 48-inch cast-iron pipe - 1380.0.
Upstream invert 36-inch cast-iron pipe - 1385.5.
Downstream invert 48-inch cast-iron pipe - 1378.6.
Downstream invert 36-inch cast-iron pipe - 1378.1.
Streambed (near outlet works) - 1368.0 (approximate).

d. Reservoir Length. (Miles.)

Normal pool - 1.1.
Maximum pool - 1.2.

e. Storage. (Acre-feet.)

Normal pool (spillway crest) - 2,115.
Maximum pool (top of dam) - 3,744.

f. Reservoir Surface. (Acres.)

Normal pool (spillway crest) - 181.
Maximum pool (top of dam) - 185 (approximate).

g. Dam.

Type - Combination homogeneous earthfill with central masonry core and masonry gravity structure.

Length - 380 feet (total, including spillways).

Height - 64 feet.

Top Width - Variable.

Side Slopes - Upstream - 1V on 3.5H.
Downstream - 1V on 2.5H.

Zoning - Homogeneous earthfill.
Central masonry core.

Cutoff - Central masonry core on concrete foundation on bedrock 10 feet below original ground. Masonry gravity structure on concrete foundation on bedrock 6 feet below original ground. Spillway on central core wall or concrete foundation upstream face down 8 feet into rock. Upstream embankment designed as cutoff.

Grout Curtain - None.

h. Diversion and Regulating Tunnel.

Type - Intake structure - 6 feet wide, 46 feet high.
Masonry tunnel - 6 feet wide, 16 feet high.
Screen chamber - 6 feet wide, 50 feet high.
Cast-iron pipe (CIP) - 48-inch diameter.
36-inch diameter.

Length - Intake structure - 5 feet.
Masonry tunnel - 55 feet.
Screen chamber - 16 feet.
48-inch CIP - 100 feet.
36-inch CIP - 200 feet.

Closure - Intake structure - none.
Masonry tunnel - four 8' x 5' wooden bulkheads.
48-inch CIP - two manually operated 48-inch diameter, 1-1/2 threads per inch (tpi), rising stem, horizontal gate valves with 5.86 to 1 exposed beveled spur and pinion gear reducers.
36-inch CIP - two manually operated 36-inch diameter, 1-1/2 tpi rising stem, horizontal gate valves with 5.86 to 1 exposed beveled gearing; one manually operated 36-inch, 2 tpi rising stem, vertical gate valve, 5.86 to 1 exposed beveled gear reducer with 1-1/2 to 1 chain drive reducer.

Access - Intake structure - none.
Tunnel, screen chamber, and cast-iron pipes - slot provided in intake structure to close tunnel portal with wooden bulkhead.

Regulating Facilities - Gate valves for 48-inch and 36-inch lines in valve chambers immediately downstream from screen chamber and 60 feet downstream from screen chamber. Gate valves for 36-inch line in gatehouse 90 feet downstream from screen chamber.

i. Spillway.

Type - Masonry gravity and concrete chute.

Length of Weirs - Masonry gravity - 153.3 feet.
Concrete chute - 136.0 feet.
Total - 298.3 feet.

Crest Elevation - Masonry gravity - 1422.5.
Concrete chute - 1422.5.

i. Spillway. (Cont'd.)

Upstream Channel - 1V on 3.5H, 1.5 feet thick, rock-lined up to weir crest.

Downstream Channel - The natural channel below the dam is straight; the left bank is protected with Derrick stone except for a short reach; and the right bank is unprotected.

j. Regulating Outlets. None other than outlet works.

SECTION 2

ENGINEERING DATA

2.1 Design.

a. Data Available. Very little engineering data was available for review for the original structures or for the 1899 and 1902 modifications. In a study performed in 1914 by the Pennsylvania Water Supply Commission, an account of design concepts, geology, construction materials and methods, and design features was prepared for the original structure from interviews with the Owner, from visual inspection, and from other sources. The 1914 study also included analyses for hydroloty, hydraulics, and stability of the principal features. Loading assumptions and summaries of results for the analyses are on file. This study and subsequent ones made in 1915 to confirm the results were the bases for the recommendations for the spillway improvements made in 1916. Stability computations, a report on a model test (Photographs I and J), and other design data are available for the concrete chute spillway that was added in 1958. The model test was for both the masonry gravity spillway and concrete chute spillway. It was performed at Alden Hydraulic Laboratory, Worcester Polytechnic Institute, Worcester, Massachusetts.

b. Design Features. The drawings indicate that the main section of the dam is a combination of earth embankment at the right abutment and masonry gravity section at the left abutment. A general plan is shown on Plate 1A. The embankment is a homogenous earthfill structure with a central masonry core wall. The bottom of the core wall is 6 to 10 feet below the original ground line. The core wall is on concrete footings which are founded on clay. The embankment section is about 160 feet long and 64 feet high at the original streambed. Embankment sections are shown on Plate 2. The masonry gravity section of dam is about 70 feet long and about 75 feet high and is founded on rock (Photograph K).

The right abutment wall ties into the masonry core wall of the embankment section and extends upstream about 1,050 feet. The wall is masonry and has a level earth backfill on the landside and a sloping earth backfill on the reservoir side. The wall projects 1 foot above the backfill. The top elevation of the wall is the same as top of dam elevation.

The left abutment wall ties into the masonry gravity section of the dam and extends upstream about 1,250 feet. Wall sections are shown on Plate 8, and the wall is also shown on Photograph A. The wall is masonry and has a level earth backfill on the landside and no fill on the reservoir side. There is a 1-foot high concrete extension on top of the wall. The wall projects 0 to 15 feet above the backfill. The top elevation of the wall is the same as top of dam.

To the right of the masonry gravity section of the dam is a masonry gravity spillway that is 153.3 feet long and 65 feet high (Photographs B and G). The left side of the spillway is founded on rock and the right side is founded on a concrete footing that was placed on clay. In 1916, the spillway was strengthened with 38 1-1/2-inch diameter, 32-foot long, twisted steel bars grouted in 3-inch diameter holes located 1.5 feet from the upstream face. The masonry gravity spillway section and strengthening details are shown on Plates 3 and 4, respectively.

To the right of the masonry gravity spillway is a concrete chute spillway with a 136-foot long crest that was constructed on the embankment section in 1958 (Photographs C and H). The masonry spillway and concrete chute spillway are separated by a 20-foot section of embankment. The spillway plan is shown on Plate 5. The concrete chute spillway has a reinforced-concrete weir with a rounded crest. The weir is tied into the masonry core wall with 1-7/8-inch diameter, prestressed anchor rods 25 feet long on 5-foot centers. Spillway details are shown on Plate 6. The chute has a variable bottom slope and curves to the left. A concrete training wall at the right side of the chute guides flow towards the main channel. Drains are provided under the chute and along the right training wall. The masonry gravity spillway and chute spillway meet 70 feet downstream from the axis of the dam.

At the right abutment, a system of ditches collects surface runoff and discharges it into the stream through a 36-inch BCCMP. A drain collects subsurface water from behind the concrete chute spillway right wall and outlets into a manhole located at the toe of the embankment. A 6-inch clay pipe connects the manhole to the 36-inch BCCMP.

The outlet works is located at the left side of the masonry gravity spillway and consists of the following main features: a masonry approach channel, a masonry intake structure, a masonry tunnel, a masonry screen chamber, and three gatehouses. A plan of the outlet works is shown on Plate 7. The outlet works are also shown on Photograph D. The piping consists of 36-inch and 48-inch cast-iron pipes. The 48-inch pipe is an emergency line for drawing down the reservoir for repairs. It is also used as a sedimentation blowoff and for low-flow augmentation. The 36-inch pipe is the water supply line with a 36-inch branch that can also be used as an emergency line or blowoff. Manually operated gate valves, located in the gatehouses, are used for controlling the discharge.

2.2 Construction.

a. Data Available. General accounts of the construction procedures are available for the original structures. Drawings, correspondence, and photographs are on file for the 1916 modifications. Drawings, correspondence, and payment records are available for the 1958 repairs and improvements.

b. Construction Considerations. Review of the available construction data did not yield any special concerns with respect to the condition of the dam.

2.3 Operation. Few formal records of operation are available. A hydrograph for the August 1955 flood, which is the flood of record for Elmhurst, is available. The dam has been inspected at irregular intervals by Commonwealth authorities since 1921 and, in recent years, annual inspections have been made by the Owner. The Owner indicated that the problems that were observed in this inspection have existed for several years.

2.4 Other Investigations. No known investigations other than those previously described were reviewed.

2.5 Evaluation.

a. Availability. Engineering data was provided by the Division of Dams and Encroachments, Bureau of Water Quality Management, Department of Environmental Resources, Commonwealth of Pennsylvania, and by the Owner, Pennsylvania Gas and Water Company. The Owner made available an engineer, dam operators, and a valve crew for information and operating demonstrations during the visual inspection. The Owner also researched his files for additional information upon request of the inspection team.

b. Adequacy. The type and amount of design data and other engineering data are limited, and the assessment must be based on the combination of available data, visual inspection, performance history, hydraulic assumptions, and hydraulic assumptions.

c. Validity. There is no reason to question the validity of the available data.

SECTION 3

VISUAL INSPECTION

3.1 Findings.

a. General. The general appearance of this project indicated that some features have deteriorated with age and are in need of repair, while other project features have been properly maintained and are in good condition.

b. Dam. The dam embankment was maintained adequately. The sod on the downstream surface was intact and well maintained. The only noticeable problems were with the surface runoff control system at the right abutment. A 40-foot long, 36-inch diameter BCCMP, which is designed to receive flow from a ditch, had flow under it. A defective pipe junction near the midpoint of the pipe has resulted in the formation of an 18-inch diameter sinkhole. There were also signs of inadequate capacity of the system. Uncontrolled overland flow has apparently produced an eroded area just beyond the toe of the embankment. This area was filled with some large stones (Photograph E). The uncontrolled flow apparently exits over the most downstream monolith of the concrete chute spillway right wall and along the ground surface parallel to the back face of the wall. There was some erosion behind the end of the wall to a maximum depth of 4 feet (Photograph E). A manhole that is located at the toe of the embankment has two pipes entering and one leaving it. A 4-inch diameter pipe, identified by the Owner as a spillway wall drain, had an accumulation of clayey soil at its outlet. The soil was cleared away and a small, clear flow was observed. Probing the pipe indicated additional accumulation of soil along its length. The second pipe entering the manhole, an 8-inch diameter pipe entering on the uphill side from the general direction of the right abutment, had an estimated discharge of 5 to 10 gpm of clear water. Its source was not determined. Outflow from the manhole entered the midpoint of the aforementioned 36-inch BCCMP.

c. Appurtenant Structures.

(1) The downstream face of the masonry gravity spillway (Photograph G) had too much flow over it to be examined in detail. However, no blocks of stone were missing. The upstream face has earthfill against it and cannot be inspected. The stone masonry and mortar joints of the spillway walls adjacent to the spillway were apparently sound. The outlet channel left wall, which is along the spillway apron, had two leaks, both near the base. Clear water was being discharged from the leakage points. An 18-inch diameter sinkhole was in the wall backfill above one of the leakage points. Near the downstream end of the wall, a portion that had been repaired with concrete had two vertical cracks and some spalling of the concrete. The cracks extend through the wall

but no differential movement was noted. The concrete apron at the bottom of the masonry gravity spillway (Photograph F) was in excellent condition, except that joint filler was missing between two slabs near the right side. Two leakage areas were on the downstream face of the short, masonry gravity section of the dam that is adjacent to the left side of the masonry gravity spillway (Photograph K). The leakage ran onto the roof of the gatehouse and then into the earthfill along the left side of the gatehouse.

(2) The concrete chute spillway (Photograph H) was in excellent condition, and it exhibited only normal, uniform wear. All monolith joints were properly sealed. However, the following observations were made in the vicinity of the second and third right wall monoliths from the downstream end: three fine diagonal cracks, wider monolith joint separation than at other monoliths, spalling at joint on back face, a 5-inch diameter hole at joint on front face at apron level that extended 2 feet back under the wall, possible differential movement indicated by 1/2-inch monolith misalignment at top of wall, and a 5-foot diameter area on the backfill 6 inches lower than adjacent areas located about 5 feet back of the middle of the second monolith. Apparently, however, none of these deficiencies has affected the satisfactory performance of the wall. Photograph L shows the location of problem area.

(3) About 300 feet from the upstream end of the right abutment wall, a 6-inch thick segment of the concrete extension about 10 feet in length was cracked and heaved.

(4) The concrete extension on the left abutment wall was disintegrated at eight locations to a maximum depth of 6 inches. About 80 linear feet of wall was affected, and steel reinforcement was exposed in some areas. Seepage on the back face of the left abutment wall was observed in the reach from the gravity section of the dam to a point 300 feet upstream. The seepage was greatest in the first 100 feet upstream, and was generally located about 3.5 feet above the level of the backfill. The water ran down the face of the wall and into the backfill. The Owner said that an 8-inch diameter wall drain was installed in 1902, and that it is connected to a pipe that discharges over the left outlet channel wall below the masonry gravity spillway onto the spillway apron. Flow was observed coming from that pipe, and it appeared that the total seepage on the back face of the left abutment wall was about equal to the pipe discharge. A wide diagonal crack was noted at an alignment offset in the abutment wall, but no differential movement was observed. Generally, the mortar joints on the left abutment wall were sound, except for the top 3 feet, where the joints were generally in poor condition.

(5) The masonry intake structure, intake tunnel and screen chamber were submerged and could not be inspected. The overhead crane beam atop the masonry intake structure was rusted. The chain hoist, trolley, and trolley beam in the screen chamber building were covered with rust. The upper and middle gatehouses had several inches

of water on the lower level. Valves and pipes had coatings of rust. The handrail around the valve chamber in the middle gatehouse was rusted and ladders into the chamber were insecure and rusty. Exposed gears had coatings of rust and little sign of lubrication. Four men took 40 minutes to open the 48-inch valve on the emergency conduit 6 inches. The 36-inch and 48-inch valves in the upper gatehouse are normally open. The 48-inch cast-iron emergency conduit showed no evidence of caulking at the inside joints near its outlet. The two 36-inch valves, which are in the lower gatehouse, were rusted and paint was scaled from the steel sections. The chain drives, gears, and stems on those valves were well lubricated. There was one handwheel of homemade construction for the two gate valves. There was water at the bottom of the lower gatehouse, but a heavy wooden floor provided easy access to these valves.

d. Reservoir Area. The slopes adjacent to the reservoir are mild. No evidence was noted of creep, rockslides or landslides. Although portions of the watershed are developed, the Owner indicated that sedimentation is not a problem. Undeveloped portions of the watershed have a hardwood cover.

e. Downstream Channel. The natural channel below the dam is straight (Photograph F). Immediately below the dam, the left bank is lined with Derrick stone, except for a 25-foot reach near the end of the outlet channel wall. The right bank is unprotected. There was no evidence of undue erosion or instability of the channel banks. The channel area was generally clear except for growth of small brush and trees along the channel banks. Examination of the brush indicated that it was cut last year. Two highway bridges across the channel, which are about 0.5 mile downstream from Elmhurst Dam, might affect tailwater depth at the dam during large floods. The reach of stream channel below Elmhurst Dam is a stocked trout stream.

3.2 Evaluation.

a. Dam. The deficiencies of the surface runoff system were not serious at the time of the inspection. Although evidence of improper functioning and inadequate capacity were observed, the damage has been of a local, noncritical nature, and results, primarily, only in a poor appearance. Complete neglect over a long period of time, however, could result in hazard to the right spillway wall or to the embankment. The accumulation of clay at the end and along the length of the spillway wall drain indicates that some migration of soil has taken place. While not appearing serious at the present time, the effectiveness of the drain might be impaired.

b. Appurtenant Structures.

(1) The two leaks that were noted along the base of the left wall along the spillway apron did not present serious hazard to the dam

at the time of the inspection. One of the leakage points has apparently resulted in the formation of an 18-inch diameter sinkhole behind the wall. This hole has been partially filled with large rock, but loss of fine materials will probably continue. The source of the leakage might be the leakage points on the downstream face of the gravity section of the dam. Timely attention to the problem would ensure that no hazardous conditions develop. The cracking along the same wall does not present significant hazard to the dam at the present time.

(2) The performance of the right spillway wall has not been affected by the problems that were observed, but it should be noted that the wall is a key feature of the project. If the situation is stable, no hazard exists. However, if the observed conditions are part of an active situation, unsatisfactory performance could result.

(3) The condition of the concrete extension on the right abutment wall is undesirable, but does not appear to present significant hazard. The quantity of flow that would go around the right end of the dam would probably not seriously erode the embankment.

(4) The condition of the concrete extension on the left wall is similar to the right abutment wall, but it is greater in extent and would allow larger quantities of water to overflow. The Owner said that a concrete apron was placed just below the backfill surface to resist erosion of the backfill if the wall were overtopped. This apron was constructed after the 1955 flood, which resulted in severe backfill erosion when the wall was overtopped. The concrete apron would now prevent such erosion. The effect of uncontrolled flow around the left abutment of the dam, however, cannot be predicted. At the least, the condition of the concrete extension is undesirable. The diagonal crack at the alignment offset was first noted in 1925 and does not appear to present significant hazard. Although not noted in previous recent inspections, the seepage through the left abutment wall near the dam appears to have existed for some time. The Owner said that the rate has been stable. Although it appears that this seepage is presently collected and handled in a safe manner by the wall drain, it is undesirable and the amount will probably increase with time.

(5) The greatest immediate concern for the outlet works is the leakage into the valve chambers of the gatehouses. Because of its location, it requires special effort to inspect, and significant change in character might develop without being readily apparent. The leakage will probably increase in time and could result in further structural deterioration. The generally inadequate maintenance of operating mechanisms could affect the functioning of the gates during emergency conditions. Similarly, hoist capacity could be reduced by corrosion. Access to the 48-inch valve is hazardous, and it is considered inadequate.

c. Reservoir Area. No conditions were observed in the reservoir area that might present significant hazard to the dam.

d. Downstream Channel. The only feature of the downstream channel that might affect the dam is the location of the highway bridges. During large floods, the bridges might cause additional buildup of tailwater, which would increase the uplift loading on the dam.

SECTION 4

OPERATIONAL PROCEDURES

4.1 Procedures. The reservoir level is maintained at spillway crest Elevation 1422.5 with excess reservoir inflow flowing over the spillways. A 36-inch diameter cast-iron pipe water supply line draws water from the reservoir at Elevation 1385.5 to gravity feed the distribution lines. The three gate valves on the water supply line are normally open; the gate valve on the branch blowoff line is closed. A 48-inch diameter cast-iron pipe blowoff line with its intake at Elevation 1380.0 can discharge water into the stream below the spillways. The upstream valve on the blowoff is normally open and the downstream valve is normally closed. The blowoff lines are not used frequently because sediment from the bottom of the reservoir makes the stream turbid. The stream is a trout stream.

4.2 Maintenance of Dam. The dam is visited daily by two caretakers who are responsible for observing the general condition of the dam and appurtenant structures and for reporting any changes or deficiencies to the Engineering Department of Pennsylvania Gas and Water Company. When the reservoir is below the spillway crest, the caretakers report the reservoir elevation to the Owner's Engineering Department. This information is used by the Engineering Department for regulating flows in the distribution system. A Pennsylvania Gas and Water Company engineer makes a formal inspection of the dam each year, and the records are kept on file and used for determining priority of repairs. Informal inspections are also made when the engineer is on the site for other reasons. The embankment is mowed at regular intervals and the brush is cut annually.

4.3 Maintenance of Operating Facilities. Although some facilities were properly maintained, some were not. It appears that there is no regular maintenance program.

4.4 Warning Systems in Effect. The Owner furnished the inspection team with a chain of command diagram for Elmhurst Dam and a generalized emergency notification list that is applicable for all the Pennsylvania Gas and Water Company dams. The Owner said that during periods of heavy rainfall, available personnel are dispatched to the dams to observe conditions. All company vehicles are equipped with radios, and the personnel can communicate with each other and with a central control facility. Evaluation of risk is made by the Owner's Engineering Department. The Owner's Engineering Department is also responsible for notification of emergency conditions to the local authorities. Detailed emergency operational procedures have not been formally established for Elmhurst Dam but are as directed by the Owner's Engineering Department.

4.5 Evaluation. Except for not opening the blowoff lines on a regular basis, the operational procedures appear to be satisfactory. Infrequent operation of the blowoff lines could affect their functioning satisfactorily during emergency conditions. The procedures used by the Owner for inspecting the dam are adequate. Some of the operating facilities are not maintained on a regular basis. In general, the warning system is adequate, but it would be more effective if it were more detailed.

SECTION 5

HYDROLOGY AND HYDRAULICS

5.1 Evaluation of Features.

a. Design Data.

(1) No hydrologic and hydraulic analyses for the original Elmhurst Dam design were available for review. The spillway capacity has been estimated several times for the various construction modifications that have evolved. Spillway capacity, as used in this Section, represents the combined capacity of the masonry gravity and concrete chute spillways.

(2) In the recommended guidelines for safety inspection of dams, the Department of the Army, Office of the Chief of Engineers (OCE), established criteria for rating the capacity of spillways. The recommended spillway design flood for the size (intermediate) and hazard potential (high) classification of Elmhurst Dam is the Probable Maximum Flood (PMF). If the dam and spillway are not capable of passing the PMF without overtopping failure, the spillway capacity is rated as inadequate. If the dam and spillway are capable of passing one-half of the PMF without overtopping failure, the spillway capacity is not rated as seriously inadequate. A spillway capacity is rated as seriously inadequate if all of the following conditions exist:

(a) There is a high hazard to loss of life from large flows downstream of the dam.

(b) Dam failure resulting from overtopping would significantly increase the hazard to loss of life downstream from the dam from that which would exist just before overtopping failure.

(c) The dam and spillway are not capable of passing one-half of the PMF without overtopping failure.

(3) In 1958, Thomas H. Wiggin, Consulting Engineer, New York City, proposed, among other recommendations, increasing the spillway capacity from 12,100 cfs to 28,700 cfs by increasing the existing masonry gravity spillway capacity and adding a concrete chute spillway. The recommended design was tested through a model study at a maximum discharge of 31,000 cfs (Photographs I and J). The modifications were accepted, and the improvements were made in 1958. Although the spillway was designed for a capacity of 28,700 cfs, in the model study it was found that 31,000 cfs could be passed without overtopping the embankment. For this study, the 31,000 cfs was accepted as the spillway capacity (Appendix C).

(4) The Elmhurst watershed is partially owned by the Pennsylvania Gas and Water Company. Some of the watershed is developed. Hydrologic analysis for this study was based on existing conditions and the effects of future development of the watershed were not considered.

b. Experience Data. For this study, a PMF peak previously calculated for hydrologically similar Stillwater Reservoir watershed was transposed to the Elmhurst watershed. In the same respect, a PMF peak previously calculated for a potential reservoir site on Fall Brook was transposed to the Curtis Reservoir watershed, which is located on a tributary of Roaring Brook with a drainage area of 2.4 square miles. Curtis Dam is located less than 2,000 feet from the normal upstream limits of Elmhurst Reservoir. Curtis Dam is shown on Photographs M and N. The PMF peak flow was estimated to be 39,930 cfs at Elmhurst and 6,270 cfs at Curtis. The Curtis component of the Elmhurst PMF peak flow is 2,570 cfs. Hydrologic computations are presented in Appendix C.

c. Visual Observation. On the date of the inspection, no conditions were observed that would indicate that the spillway capacity would be significantly reduced during a flood occurrence.

d. Overtopping Potential. Two cases were analyzed to check the overtopping potential. Case 1 considered a storm over the entire Elmhurst watershed. Case 2 considered a storm over the Curtis watershed alone. In both cases, the effects of Lake Henry and the breached Hollister Dam, both upstream of Elmhurst Dam, were considered to be negligible.

(1) For Case 1, the Curtis component of the Elmhurst PMF was routed through the Curtis Reservoir, and the outflow was added to the contribution from the rest of the Elmhurst watershed. Based on the total time of hydrograph that is used, the spillway capacity of 2,260 cfs and surcharge storage effect of Curtis Dam might or might not pass the Curtis component of the Elmhurst PMF without overtopping Curtis Dam (Appendix C). A detailed analysis of this complex hydrological system is beyond the scope of the Phase I hydrology and hydraulics analyses. Neglecting the Curtis watershed completely, the PMF peak inflow into Elmhurst would be 37,360 cfs (39,930 - 2,570). This peak inflow of 37,360 cfs into Elmhurst Reservoir is greater than the spillway capacity of Elmhurst Dam. A check of the surcharge storage effect of Elmhurst Reservoir shows that the surcharge storage available is insufficient to contain an inflow with a peak flow of 37,360 cfs without overtopping the dam (Appendix C). If Elmhurst Dam should fail because of overtopping, the potential for failure of No. 7 Dam, located downstream of Elmhurst Dam, is greatly increased. A Phase I Inspection Report on No. 7 Dam indicated that No. 7 Dam would be overtopped before Elmhurst Dam.

(2) For Case 2, the Curtis PMF (6,270 cfs) was routed through the Curtis Reservoir, assuming that the Elmhurst pool elevation is initially at the normal pool (spillway crest) and that inflow into Elmhurst Reservoir other than the Curtis contribution is negligible. The peak inflow of 6,270 cfs is greater than the spillway capacity of 2,260 cfs of Curtis Dam. A check of the surcharge storage effect of Curtis Reservoir shows that the surcharge storage available is insufficient to contain an inflow with a peak flow of 6,270 cfs without overtopping the dam. One-half of the PMF of Curtis watershed is 3,135 cfs and is greater than the spillway capacity of 2,260 cfs. A check of the surcharge storage effect of Curtis Reservoir shows that the surcharge storage available is insufficient to contain an inflow with a peak flow of 3,135 cfs without overtopping the Curtis Dam. An estimate of the peak of the failure hydrograph of Curtis Dam was made and was found to be 82,000 cfs (Appendix C). If Curtis Dam fails because of overtopping, the spillway capacity and surcharge storage effect of Elmhurst Dam are sufficient to contain an inflow with a peak of 82,000 cfs without overtopping the dam, providing that the inflow from the remaining 34.9 square miles of watershed is negligible. The surcharge storage effect method used was based on the Phase I hydrology and hydraulics procedure, which disregards waves or surges that might produce splashover.

e. Downstream Conditions. As shown on Plate 1, approximately 30 structures are located along Roaring Brook between Elmhurst Dam and No. 7 Dam that would be affected by a failure of Elmhurst Dam. The nearest structures are located about 0.5 mile downstream from Elmhurst Dam in the Village of Elmhurst. Also, bridges carrying U.S. Route 611 and a local road in the Village of Elmhurst would be affected, as well as the Erie-Lackawanna Railroad tracks that are along the valley of Roaring Brook. No. 7 Dam is located about 6.5 miles downstream from Elmhurst Dam. If Elmhurst Dam should fail due to overtopping, the potential for failure of No. 7 Dam would be greatly increased if No. 7 Dam had not failed previously due to overtopping. Approximately 1 mile below No. 7 Dam, the Erie-Lackawanna Railroad tracks cross the stream. The urban areas of Dunmore and Scranton are located about 1.3 miles below No. 7 Dam and would be affected by a large flow. The downstream conditions indicate that a high hazard classification is warranted for Elmhurst Dam.

f. Spillway Adequacy.

(1) The spillway will not pass the PMF without overtopping the dam. Therefore, based on OCE criteria as outlined in Paragraph 5.1a(2), the spillway capacity of Elmhurst Dam is rated as inadequate. One-half of the PMF of the Elmhurst watershed is 19,965 cfs and is considerably less than the 31,000 cfs spillway capacity. Therefore, based on OCE criteria as outlined in Paragraph 5.1a(2), the spillway capacity of Elmhurst Dam is not rated as seriously inadequate. For Curtis Dam, considering the effects of the surcharge storage of 288 acre-feet, the Curtis spillway discharge capacity of 2,260 cfs can accommodate a flood

with a peak inflow of 2,580 cfs for a storm of the same duration as the Curtis PMF. This is 41 percent of the Curtis PMF peak inflow. Considering the effects of the combined Elmhurst Reservoir and Curtis Reservoir surcharge storage of 1,934 acre-feet, the Elmhurst spillway discharge capacity of 31,000 cfs can accommodate a flood with a peak inflow of 32,125 cfs for a storm of the same duration as the Elmhurst PMF. This is 80 percent of the Elmhurst PMF.

(2) The maximum tailwater is estimated to be Elevation 1380 at the spillway capacity of 31,000 cfs. At maximum pool elevation, there is a difference of about 50 feet between headwater and tailwater. If Elmhurst Dam should fail due to overtopping during the PMF, the hazard to loss of life downstream from the dam will be significantly increased from that which would exist just prior to overtopping.

SECTION 6

STRUCTURAL STABILITY

6.1 Evaluation of Structural Stability.

a. Visual Observations.

(1) General. The visual inspection of the dam resulted in several observations relevant to structural stability. These observations are listed herein for the various features.

(2) Embankment. Improper functioning of the surface runoff control system at the right abutment of the dam was observed. An accumulation of clayey soil at the outlet of the concrete spillway wall drain was also noted. Detailed descriptions and evaluations of the conditions are in Paragraphs 3.1.b. and 3.2.a., respectively.

(3) Appurtenant Structures. Two leakage points at the base of the left outlet channel wall along the masonry gravity spillway apron were observed, and two others were noted on the downstream face of the masonry gravity section of the dam. Possible movement of the concrete chute spillway right wall near its downstream end was observed. Leakage was noted on the back face of the left abutment wall, and the mortar joints in the top 3 feet were generally in poor condition. Standing water was observed at the bottom of the two upper gatehouses. Detailed descriptions and evaluations of the conditions are in Paragraphs 3.1.c. and 3.2.b., respectively.

b. Design and Construction Data. No records were reviewed of design data or stability computations for the original structures built in 1889 or for the modifications made in 1899 and 1902. However, stability analyses made in 1914 and 1915 by the Water Supply Commission of Pennsylvania are on file, and the analyses were used as the bases for modifications that were made to the dam in 1916. Included in these computations are stability analyses for the masonry gravity spillway and for the two abutment walls. Improvements made in 1916 included lowering the spillway crest to increase its capacity, installing steel reinforcement near the upstream face of the spillway to strengthen the upper section, and raising and strengthening the abutment walls. Based on correspondence in the files, the work was accomplished to the satisfaction of the Commission, and, as far as is known, the spillway strengthening is still effective.

Stability computations for the concrete chute spillway that was built in 1958 are also on file. Correspondence on file indicated that the Water Power and Resources Board, Division of Dams and Encroachments, Commonwealth of Pennsylvania, reviewed the computations and that the permit application requirements were satisfied. In 1961,

the final inspection was performed, and a memorandum on file indicates that the concrete chute spillway was completed to the satisfaction of the Division of Dams and Encroachments.

For this study, stability analyses were made for the masonry gravity spillway and for the left abutment wall, which are the two features that were judged most likely not to meet OCE criteria for stability. The analyses used water at maximum pool level, and only the bottom of each section was considered. For both structures, the toe pressure and sliding factor are within acceptable limits. The resultant is outside the middle third, but it is within the base. OCE guidelines on overturning recommend that the resultant be within the middle third. Although the resultants for both structures are outside the middle third, they are within the base, and considering that the toe pressures are within acceptable limits, the resultants being outside the middle third is not considered to be a significant deviation from the recommended guidelines. Experience data is also available to confirm this evaluation as discussed in the next paragraph.

c. Operating Records. Based on the operating records, there is no evidence that any stability problems have occurred for the abutment walls, embankment, masonry gravity spillway, concrete chute spillway, or the masonry gravity portion of the dam. During the August 1955 flood, water was within 6 inches of the present maximum pool elevation, and damages were incurred to the spillway apron and behind the left abutment wall, which was overtopped by about 6 inches. However, the damages were not related to what is considered as stability problems.

d. Post-Construction Changes. As noted herein, there is adequate information on file concerning modifications made to Elmhurst Dam after 1916.

e. Seismic Stability. Elmhurst Dam is located in Seismic Zone 1. Normally, it can be considered that if a dam in this zone is stable under static loading conditions, it can be assumed safe for any expected earthquake loading. However, since there is the potential of earthquake forces moving or cracking the masonry core wall, the theoretical seismic stability of this dam cannot be assessed.

SECTION 7

ASSESSMENT, RECOMMENDATIONS, AND REMEDIAL MEASURES

7.1 Dam Assessment.

a. Safety.

(1) Based on the visual inspection, available records, calculations and past operational performance, Elmhurst Dam is judged to be in good condition. However, some maintenance and repair deficiencies were noted. A summary of features and observed deficiencies are listed below:

Feature and Location	Observed Deficiencies
Surface Runoff Control System:	
Area adjacent to embankment	Flow under pipe. Defective pipe junction. Uncontrolled flow. Eroded areas.
Manhole	Soil accumulated at end of concrete chute spillway wall drain in manhole.
Gravity Section of Dam:	
Downstream face	Leakage.
Masonry Gravity Spillway:	
Outlet channel left wall	Leakage; sinkhole; vertical cracks.
Concrete	Loss of monolith joint filler.
Concrete Chute Spillway:	
Right wall (near downstream end)	Possible movement; 5-inch diameter hole at base; cracking.
Right Abutment Wall:	
Near upstream end	Concrete extension cracked and heaved.

Feature and Location	Observed Deficiencies
Left Abutment Wall:	
Various locations	Concrete extension disintegrated.
Near downstream end	Leakage.
Change in alignment	Diagonal crack.
Top 3 feet	Masonry joints in poor condition.
Outlet Works:	Lack of regular maintenance; hazardous access.
	Leakage into gatehouses.
Downstream Channel:	Bridges obstruct channel.

(2) The overtopping potential analysis shows that Elmhurst Dam will be overtopped by the PMF. Therefore, based on the criteria (OCE Guidelines), the spillway is rated as inadequate. Hydrologic and hydraulic analyses show that one-half the PMF peak inflow is less than the spillway capacity, therefore, based on the criteria (OCE Guidelines), the spillway capacity is not rated as seriously inadequate. The existing spillway can accommodate a flood with a peak inflow of 80 percent of the PMF peak inflow. Additional analyses for the overtopping potential of Elmhurst Dam included consideration of the hydrologic and hydraulic effects of Curtis Dam, which is located on White Oak Run about 1.4 miles upstream from Elmhurst Dam. Results of the analyses show that Curtis Dam will be overtopped by one-half the PMF (storm over Curtis watershed alone). A failure hydrograph of Curtis Dam was made, and it was found that if Curtis Dam failed, the spillway capacity and surcharge storage effect of Elmhurst Dam are sufficient to contain the peak inflow of the failure hydrograph without overtopping the dam, if inflow from the remaining drainage area above Elmhurst Dam is negligible.

(3) Review of stability computations that are on file and computations performed for this study indicate that the masonry gravity spillway and the left abutment wall are apparently structurally adequate for the maximum pool condition. For the maximum pool condition, computations show that the resultants are outside the middle third, but within the base, and that sliding factors and toe pressures are within acceptable limits. Experience data is available to confirm the stability of these sections. During the August 1955 flood, water was within 6 inches of the present maximum pool elevation.

(4) Computations for the stability of the concrete chute spillway that was added in 1958 were not reviewed. Correspondence on file indicated that a review of the computations was made by the Water Power and Resources Board, Division of Dams and Encroachments, Commonwealth of Pennsylvania, and that the computations satisfied permit application requirements. In 1961, a final inspection of the construction work was made and a memorandum on file indicates that the concrete chute spillway was completed to the satisfaction of the Division of Dams and Encroachments.

b. Adequacy of Information. The information available is such that an assessment of the condition of the dam can be inferred from the combination of visual inspection, past performance, computations performed prior to and as a part of this study, and other information.

c. Urgency. The recommendations in Paragraph 7.2 should be implemented as soon as practical or in a timely manner as noted.

d. Necessity for Further Investigations. In order to accomplish some of the remedial measures outlined in Paragraph 7.2, further investigations will be required.

7.2 Recommendations and Remedial Measures.

a. Due to the potential for overtopping of the dam, the following measure is recommended to be undertaken by the Owner as soon as practical:

(1) Develop a detailed emergency operation and warning system for Elmhurst Dam.

b. In order to correct operational, maintenance and repair deficiencies and to more accurately determine the condition of the dam, the following measures are recommended to be undertaken by the Owner in a timely manner:

(1) Repair stone masonry to eliminate leakage through downstream face of gravity section of dam, gatehouse walls, and left abutment wall. Repair deteriorated stone masonry along top 3 feet of left abutment wall.

(2) Operate the valves on the gated outlets periodically to ensure they will be functional during emergency conditions. Lubricate the operating mechanisms and repair or replace hazardous access facilities.

(3) Repair concrete extensions on right abutment wall and left abutment wall.

(4) Monitor right wall of concrete chute spillway for movement.

(5) Evaluate effect of possible improper functioning of concrete chute spillway wall drain with respect to design function. If drain must be functional, install several observation wells in the drain to determine the extent of hydrostatic pressure on the wall and to determine if repairs to the drain are necessary. If installed, monitor wells to see if condition worsens.

(6) Perform repairs on surface runoff control system, outlet channel left wall, concrete apron joint below masonry gravity spillway, and hole at base of concrete chute spillway right wall.

c. The following measures are recommended to be undertaken by the Owner when the need arises:

(1) Provide round-the-clock surveillance of Elmhurst Dam during periods of unusually heavy rains.

(2) When warnings of a storm of major proportions are given by the National Weather Service, the Owner should activate his emergency operation and warning system procedures.

SUSQUEHANNA RIVER BASIN

ROARING BROOK, LACKAWANNA COUNTY

PENNSYLVANIA

ELMHURST DAM

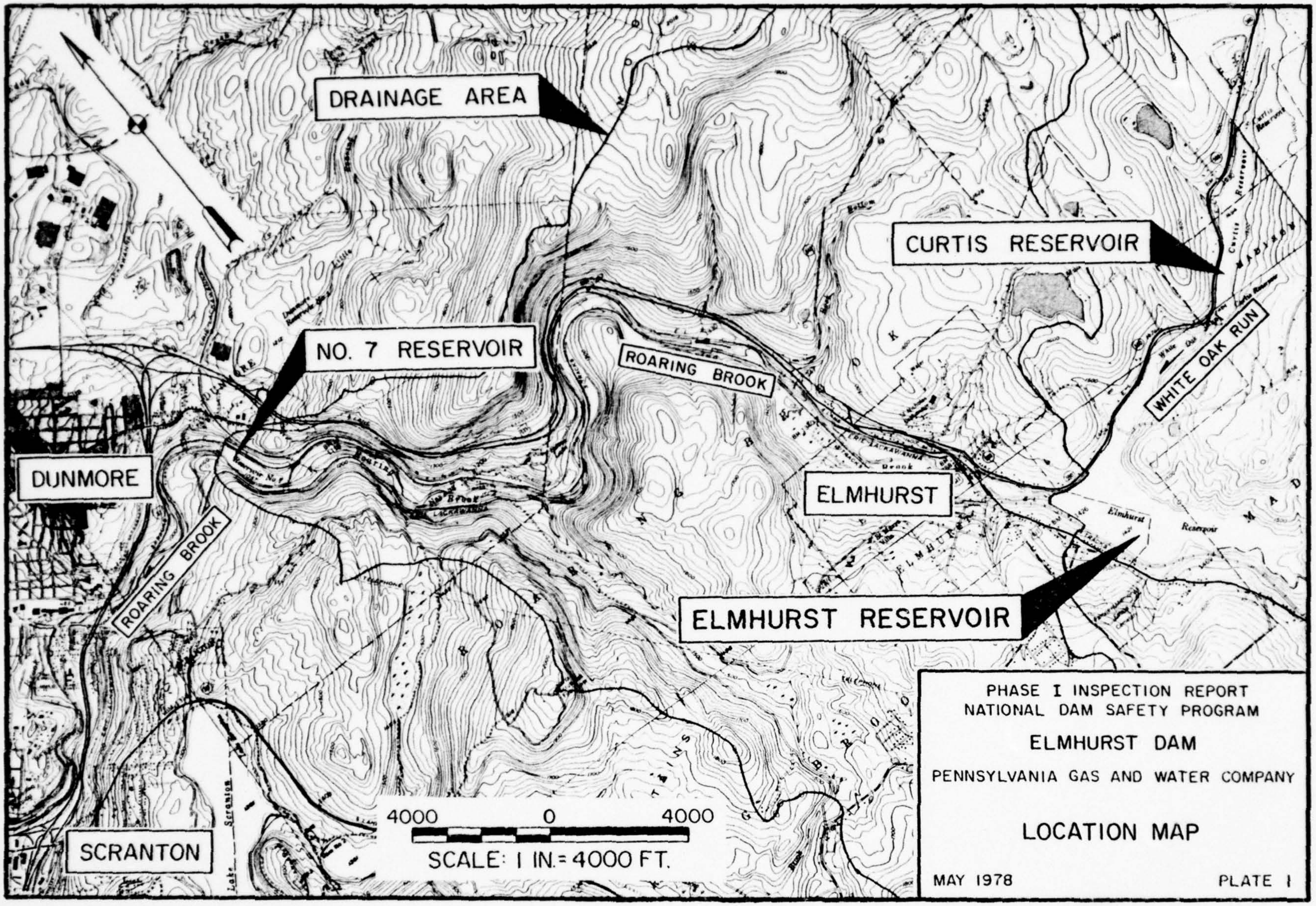
PENNSYLVANIA GAS AND WATER COMPANY

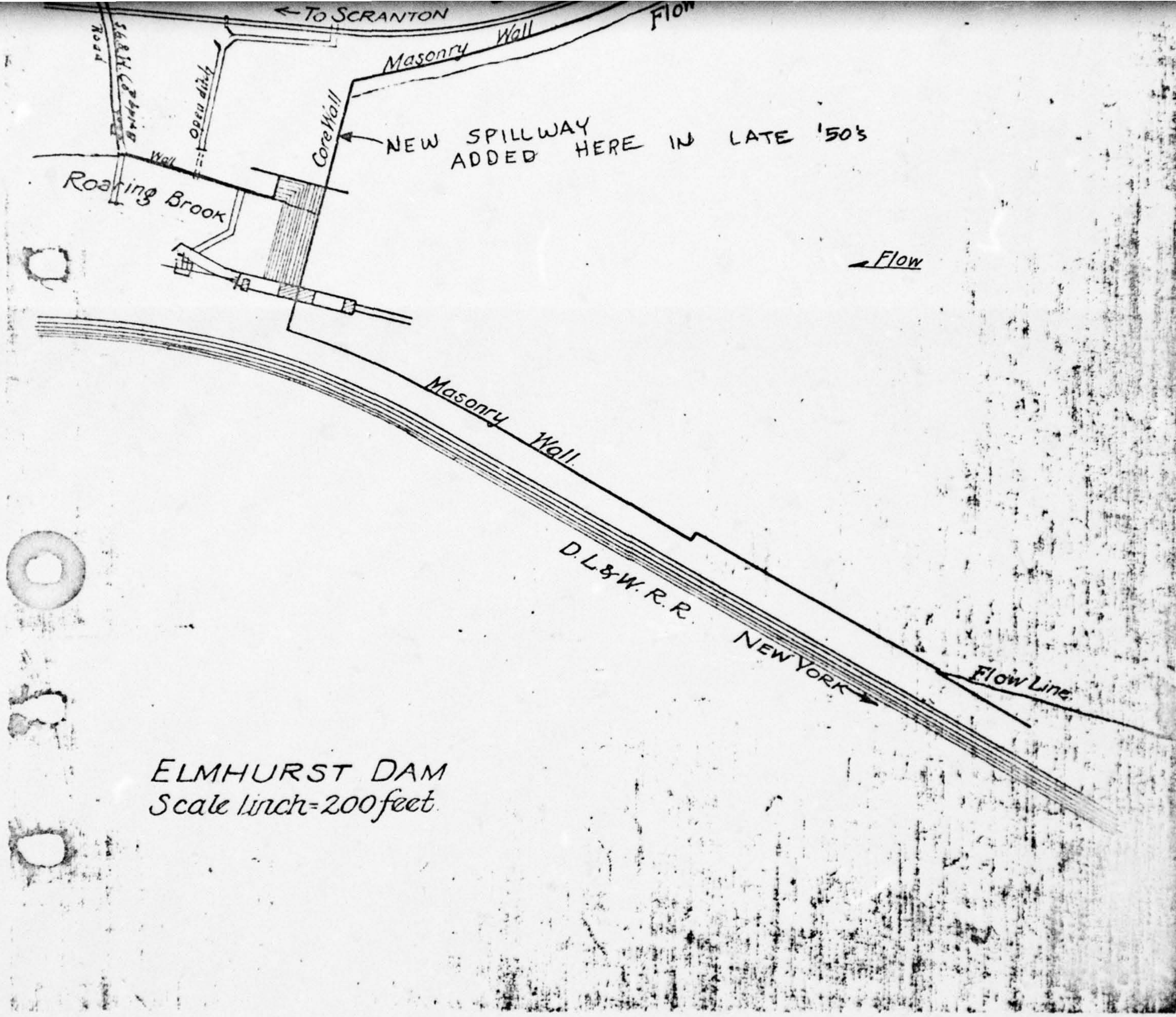
PHASE I INSPECTION REPORT

NATIONAL DAM INSPECTION PROGRAM

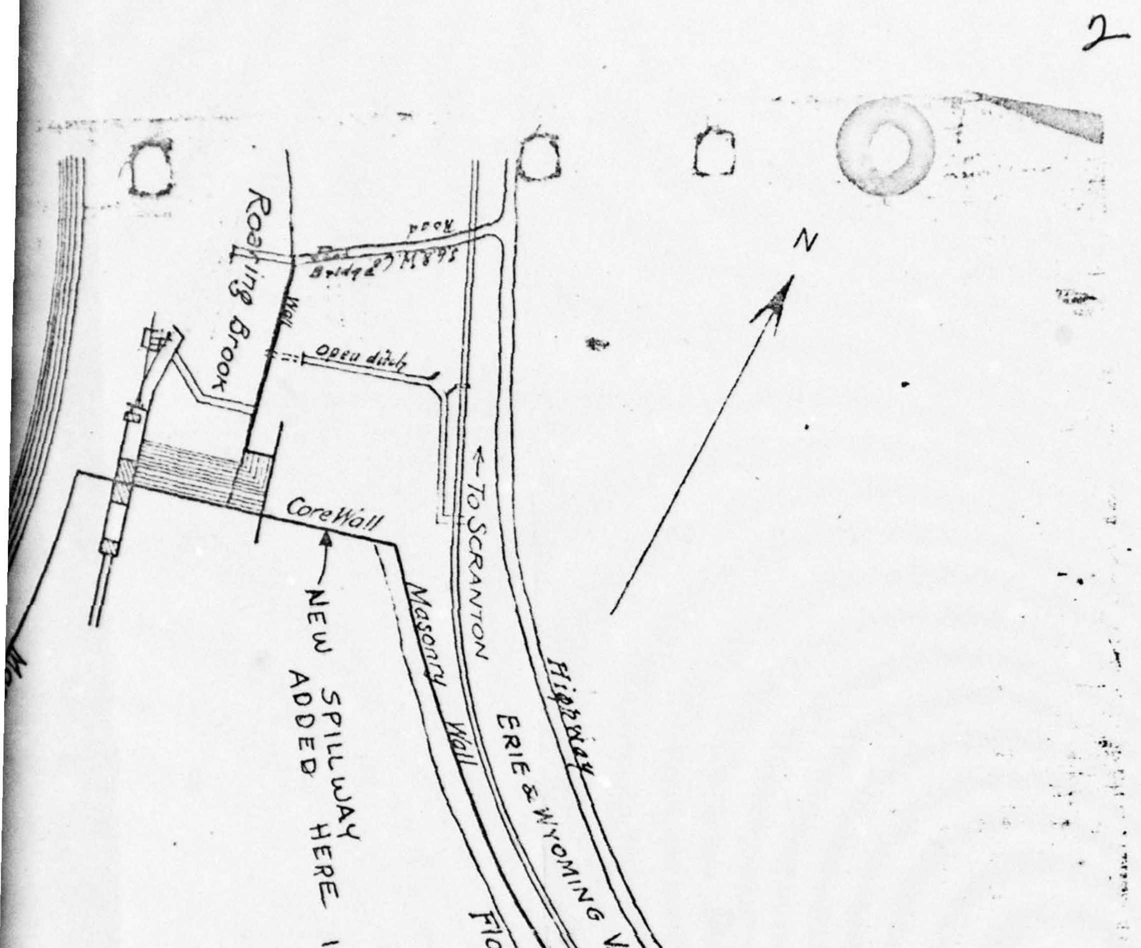
MAY 1978

PLATES


DRAINAGE AREA
CURTIS RESERVOIR
NO. 7 RESERVOIR
ROARING BROOK
WHITE OAK RUN
DUNMORE
ELMHURST
ROARING BROOK
ELMHURST RESERVOIR
SCRANTON
4000 0 4000
SCALE: 1 IN.= 4000 FT.
PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
ELMHURST DAM
PENNSYLVANIA GAS AND WATER COMPANY
LOCATION MAP
MAY 1978
PLATE 1



← To SCRANTON
Flow
Masonry Wall
Road
Bridge
Open ditch
Wall
Core Wall
NEW SPILLWAY ADDED HERE IN LATE '50's
Roaring Brook
Flow
Masonry Wall
D.L.&W. R.R. NEW YORK
Flow Line
ELMHURST DAM
Scale 1inch=200feet



PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

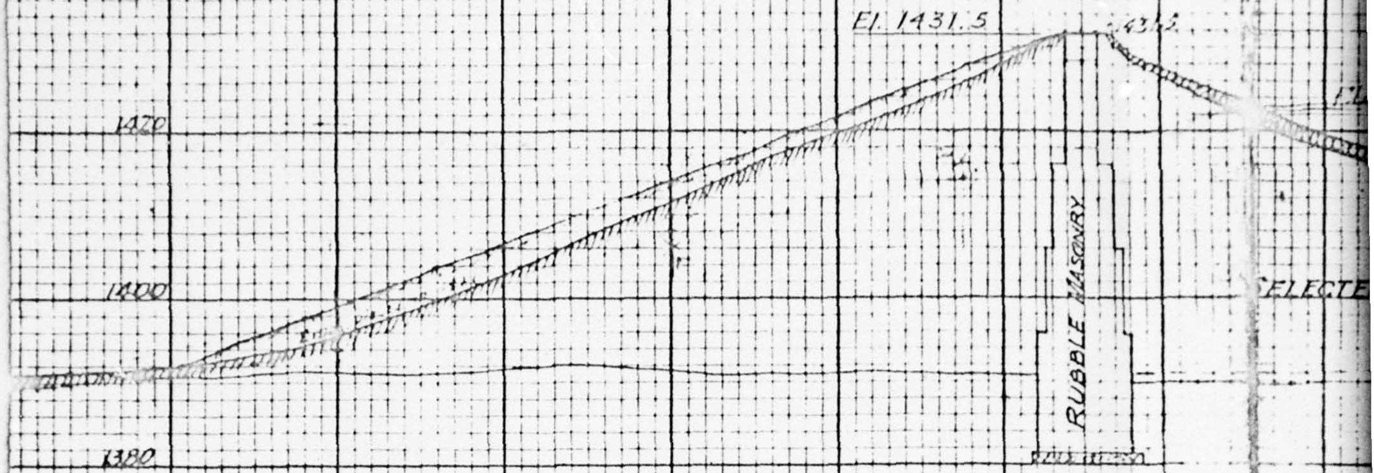
ELMHURST DAM

PENNSYLVANIA GAS AND WATER COMPANY

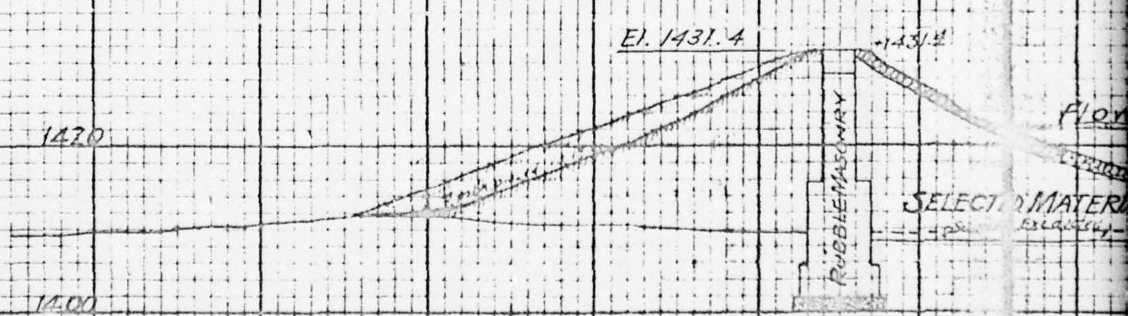
GENERAL PLAN
(PRIOR TO 1958)

MAY 1978 PLATE IA

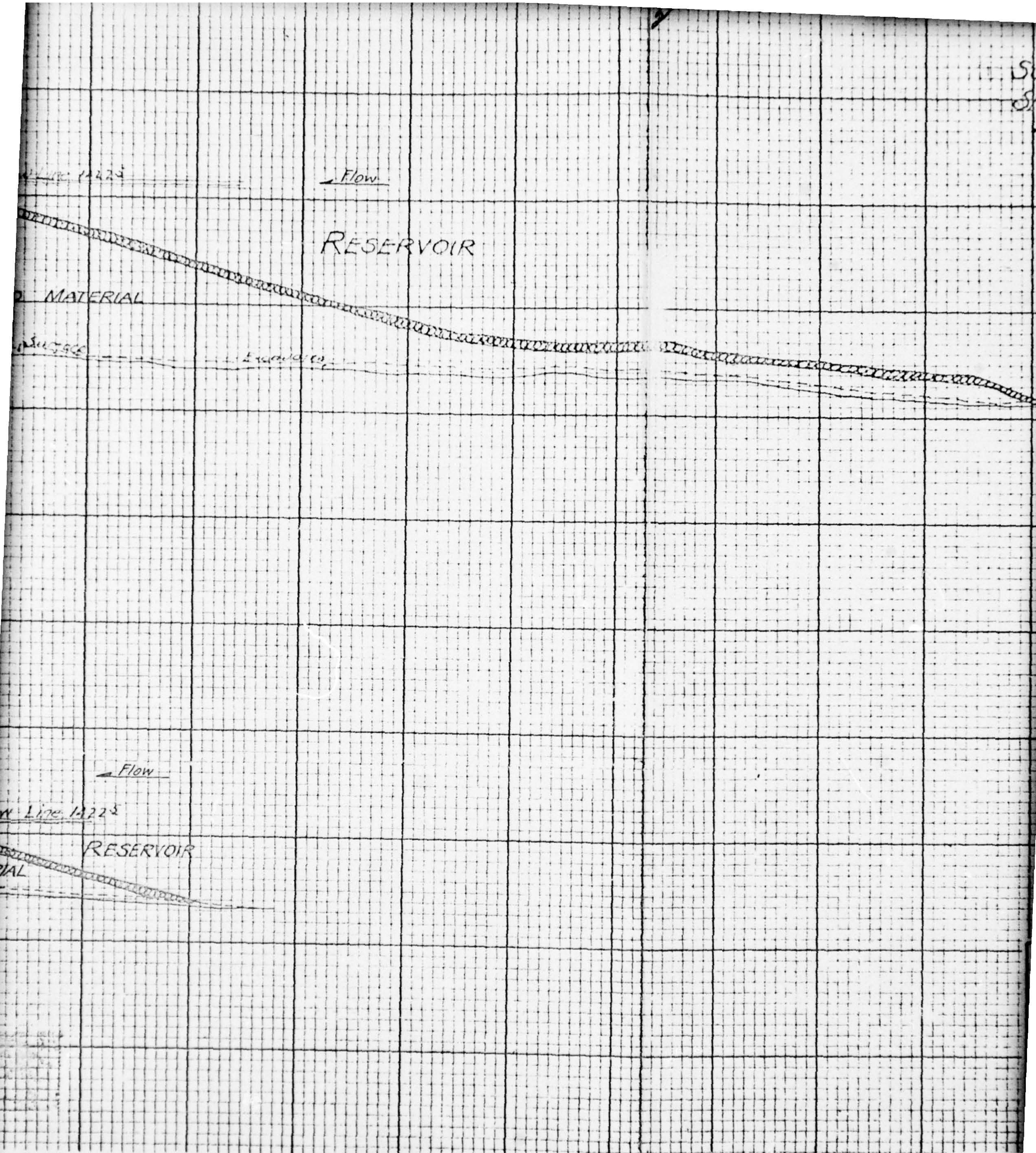
1

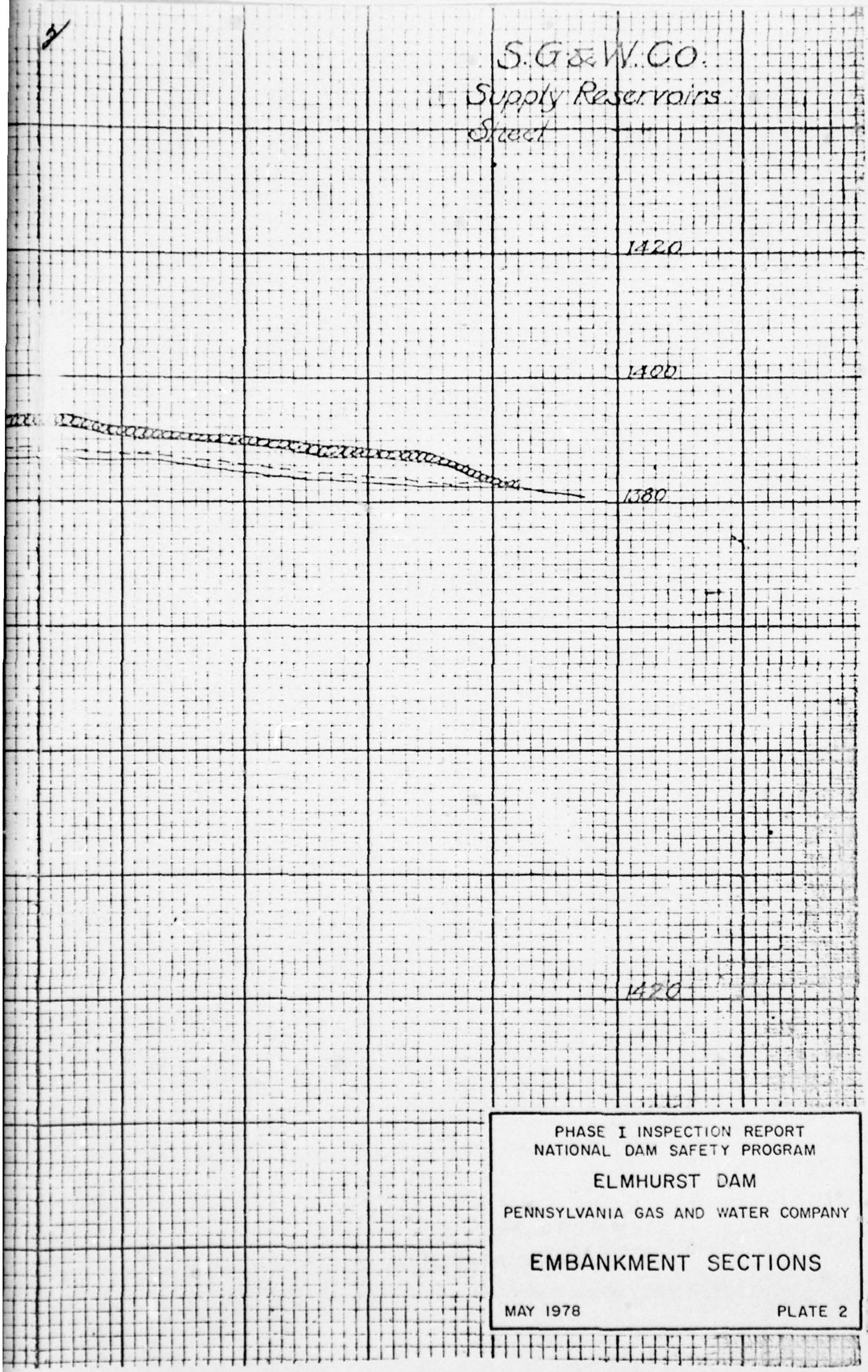


SECTION THROUGH COREWALL AND EMBANKMENT
AT STATION 17
Scale 1 inch = 20 feet



ELMHURST DAM
SECTION THROUGH CORE WALL AND EMBANKMENT
AT STATION 22
Scale 1 inch = 20 feet


Flow
RESERVOIR
MATERIAL
Flow
RESERVOIR



PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

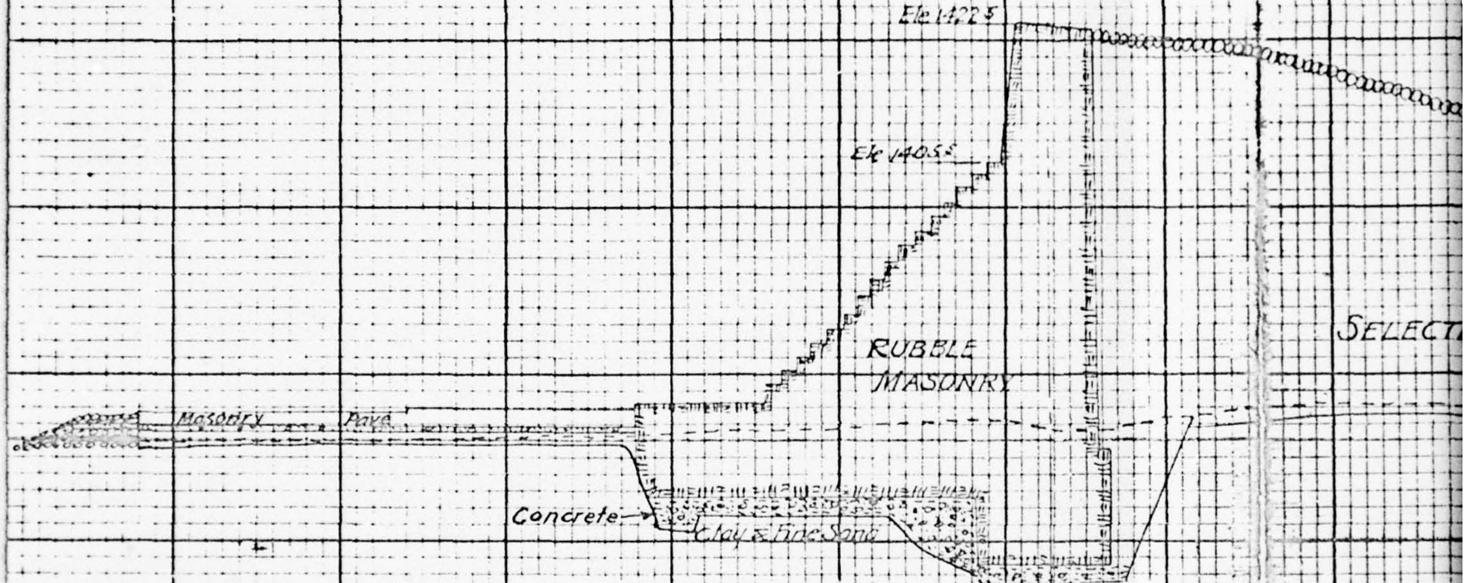
ELMHURST DAM

PENNSYLVANIA GAS AND WATER COMPANY

EMBANKMENT SECTIONS

MAY 1978 PLATE 2

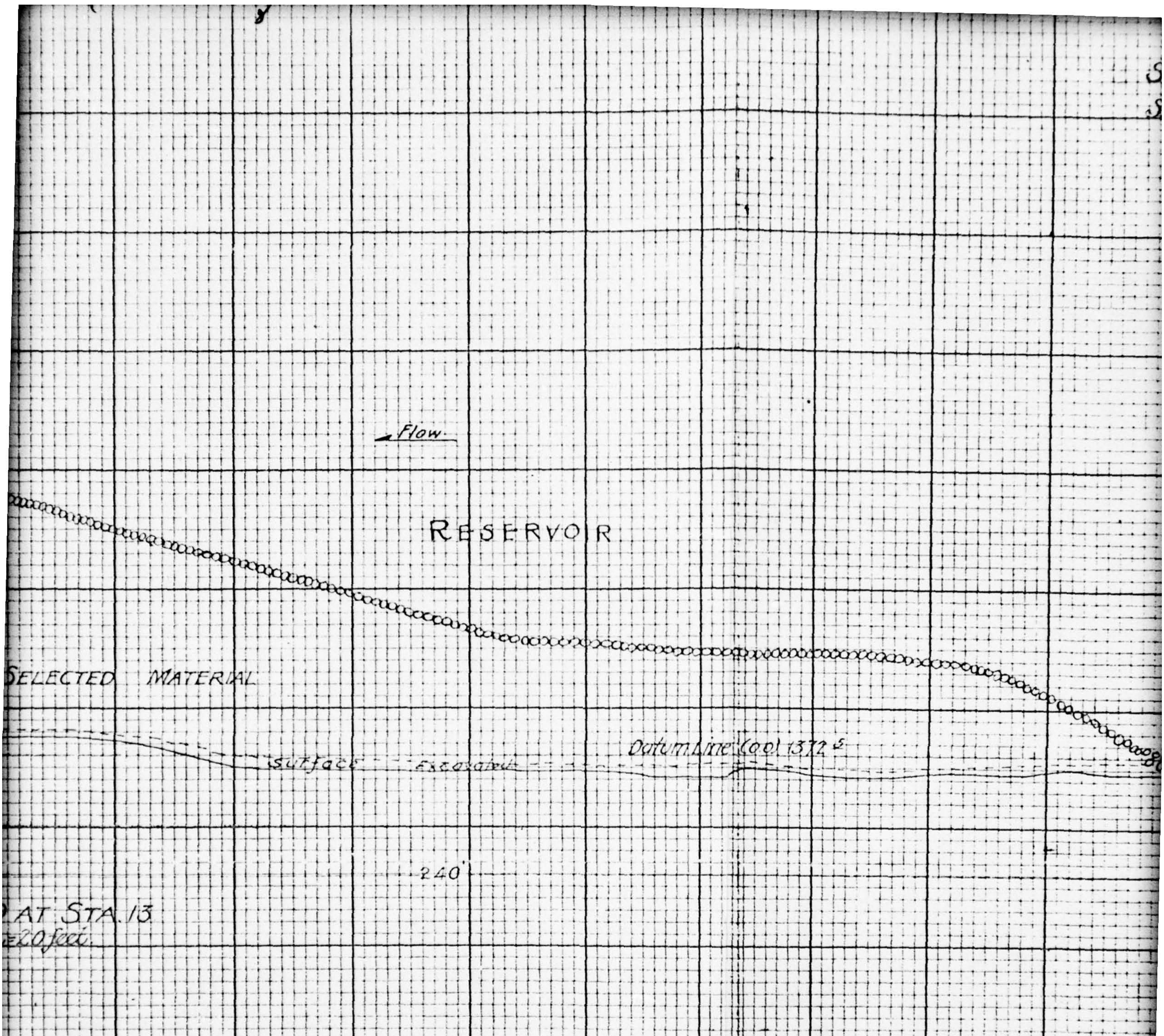
1

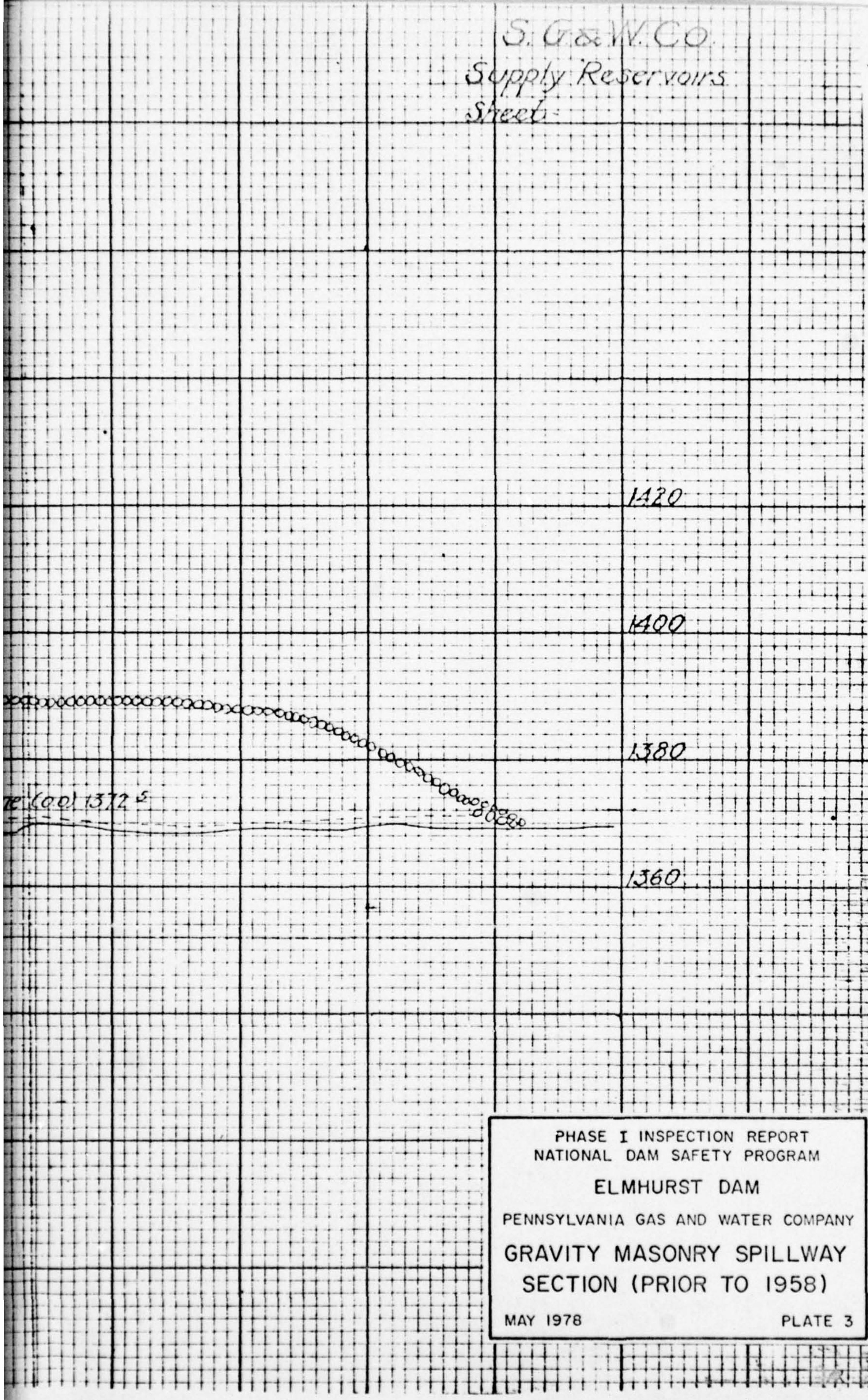


SECTIO C-D AT S

Scal 1inch=20 feet

ELMHURST DAM
SECTION THROUGH SPILLWAY
Scale 1inch=20 feet


Flow
RESERVOIR
SELECTED MATERIAL
Surface Excavated
Datum Line (Co.) 1372.5
240
AT STA. 13
=20 feet.



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NATIONAL DAM SAFETY PROGRAM

ELMHURST DAM

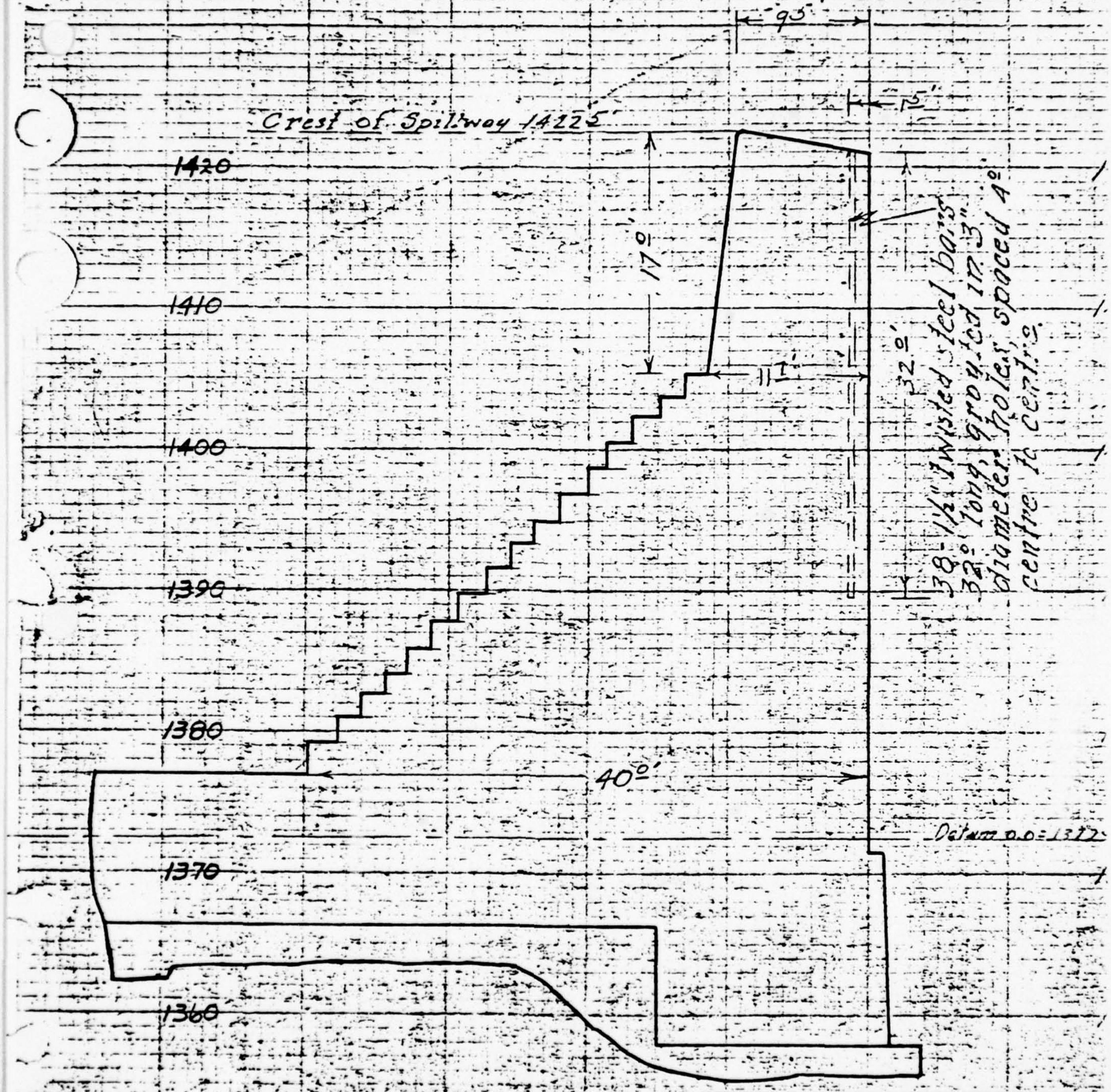
PENNSYLVANIA GAS AND WATER COMPANY

GRAVITY MASONRY SPILLWAY SECTION (PRIOR TO 1958)

MAY 1978

PLATE 3

S.G. & W. 6
Supply Reservoi
Sheet



ELMHURST DA...

Section through spi... showing method of re... spillway.

Scale 1"=1'

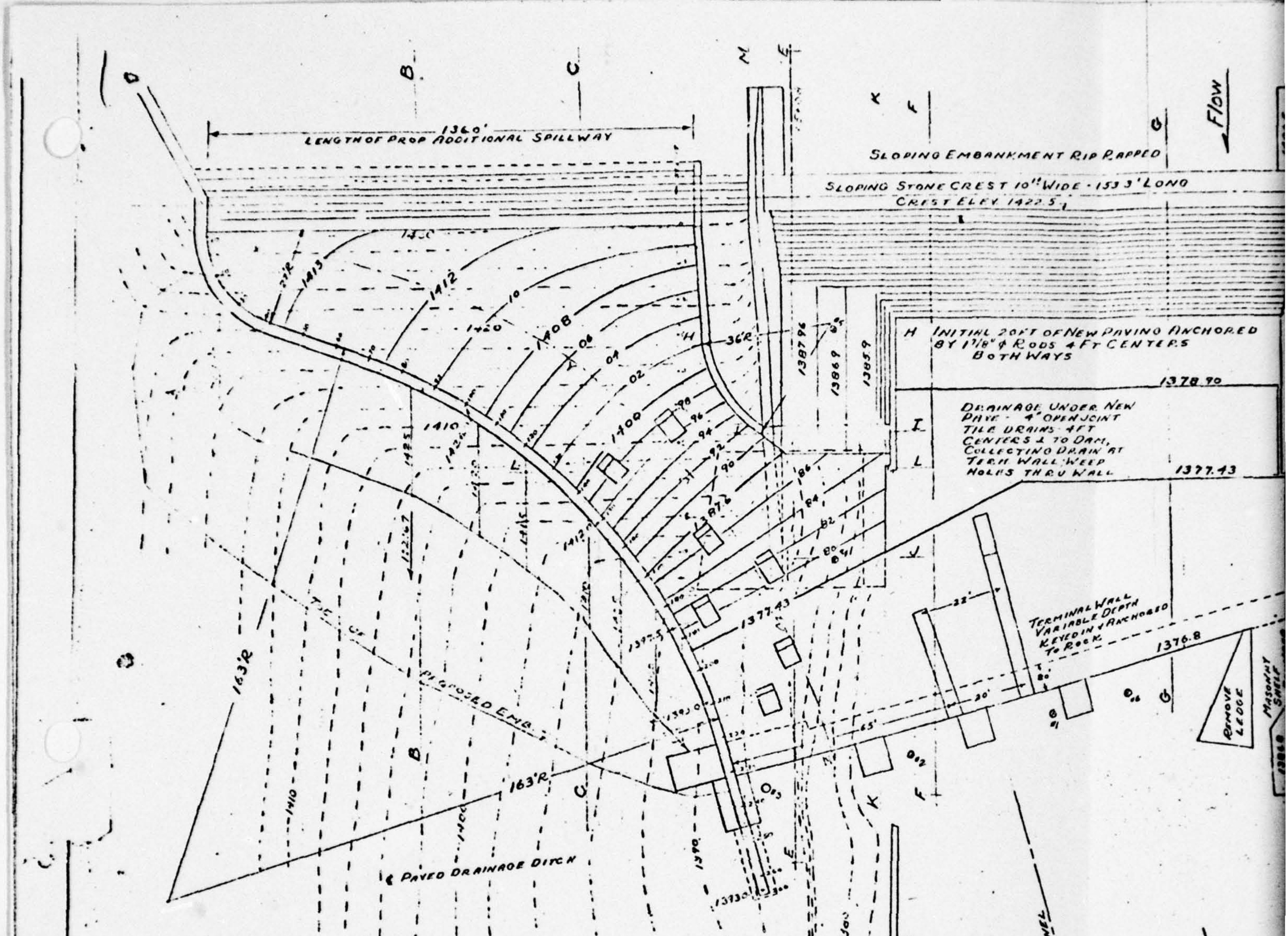
PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

ELMHURST DAM

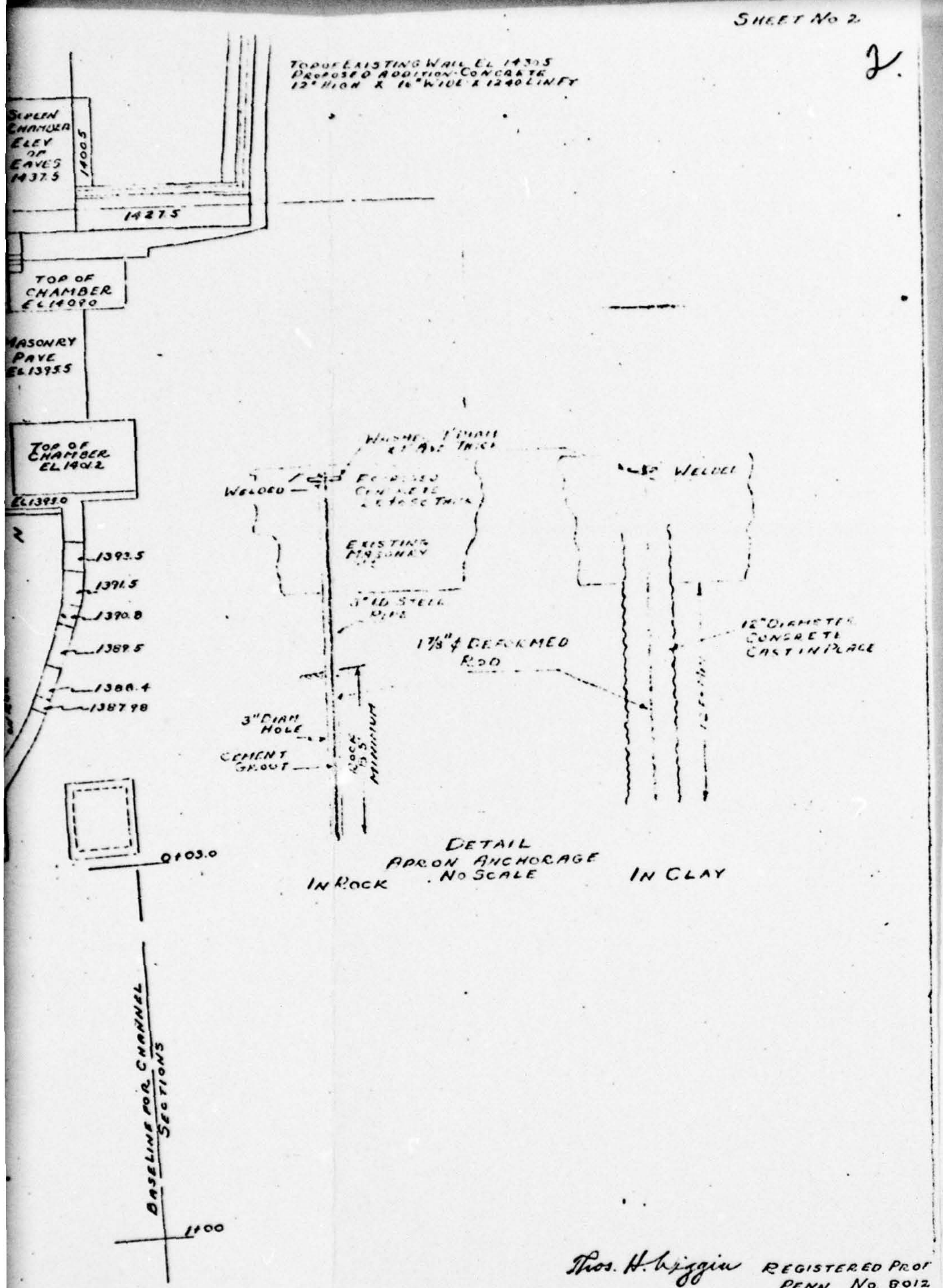
PENNSYLVANIA GAS AND WATER COMPANY

DETAILS OF STRENGTHENING SPILLWAY (1916)

MAY 1978 | PLATE 4



1360'
LENGTH OF PROP. ADDITIONAL SPILLWAY
SLOPING EMBANKMENT RIP RAPPED
SLOPING STONE CREST 10" WIDE - 153.3' LONG
CREST ELEV. 1422.5
INITIAL 20 FT OF NEW PAVING ANCHORED BY 1 1/8" ϕ RODS 4 FT CENTERS BOTH WAYS
1378.90
DRAINAGE UNDER NEW PAVE - 4" OPEN JOINT TILE DRAINS 4 FT CENTERS ⊥ TO DAM, COLLECTING DRAIN AT TERM WALL-WEEP HOLES THRU WALL
1377.43
TERMINAL WALL VARIABLE DEPTH KEYED IN + ANCHORED TO ROCK
1376.8
REMOVE LEDGE
MASONRY SHELF
FLOW
163'R
36'R
PROPOSED EMB.
℄ PAVED DRAINAGE DITCH
STONE MASONRY WALL TO BE REMOVED
PROPOSED ℄ OF CHANNEL



Thos. H. Wiggin REGISTERED PROF
PENN No 6012

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

ELMHURST DAM

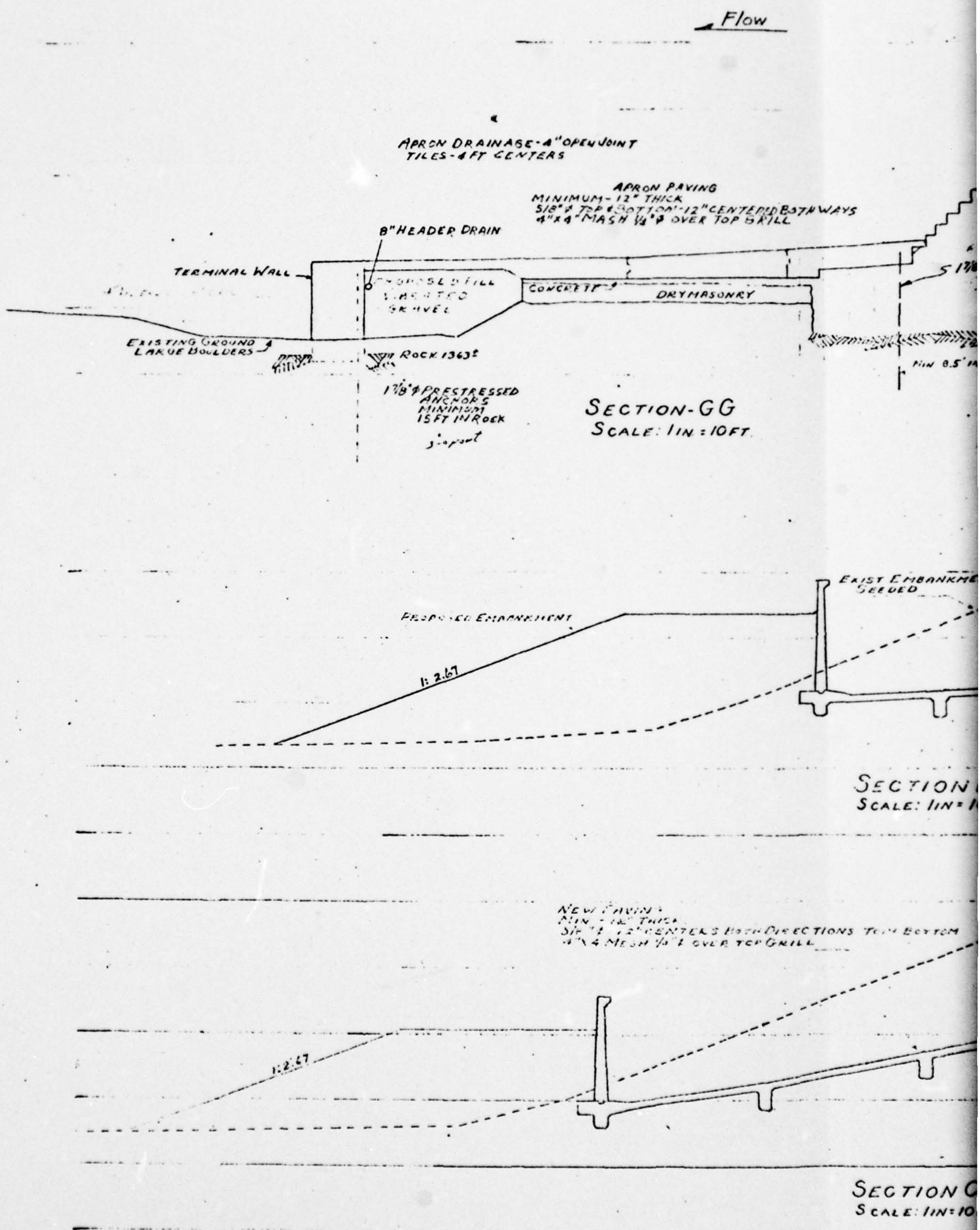
PENNSYLVANIA GAS AND WATER COMPANY

PLAN
MASONRY GRAVITY SPILLWAY
AND CONCRETE CHUTE SPILLWAY

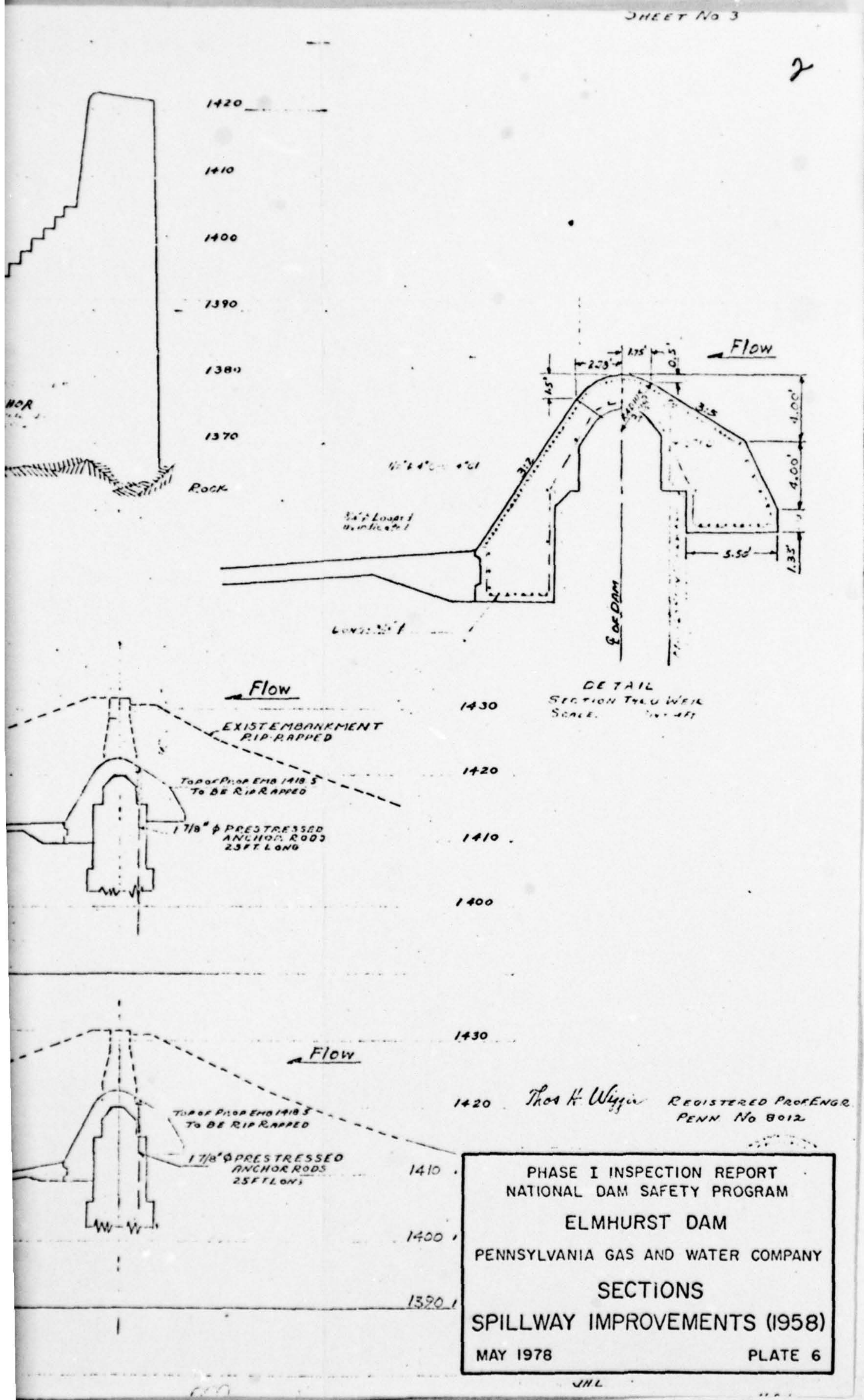
MAY 1978 PLATE 5

SERVICE

EXISTING SPILLWAY
T.H. WIGGIN CONSULTA
DRAWN BY J.H.L.
CHECKED BY



Flow
APRON DRAINAGE-4" OPEN JOINT TILES-4 FT CENTERS
APRON PAVING
MINIMUM-12" THICK
4"x4" MASH 1/4" OVER TOP GRILL
8" HEADER DRAIN
TERMINAL WALL
CONCRETE
DRYMASONRY
EXISTING GROUND
ROCK 1363±
1 7/8" PRESTRESSED ANCHORS MINIMUM 15 FT IN ROCK
SECTION-GG
SCALE: 1 IN = 10 FT
1: 2.67
SEEDED
SECTION
1:2.67
SECTION



PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

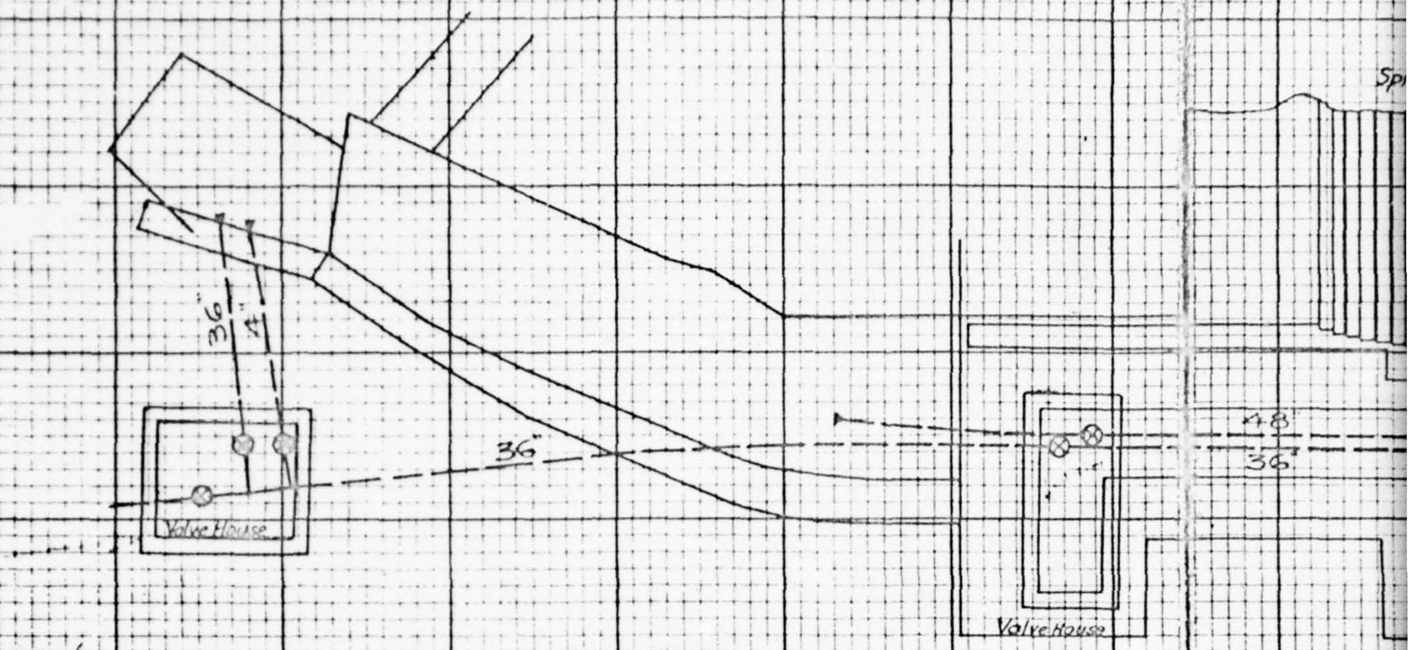
ELMHURST DAM

PENNSYLVANIA GAS AND WATER COMPANY

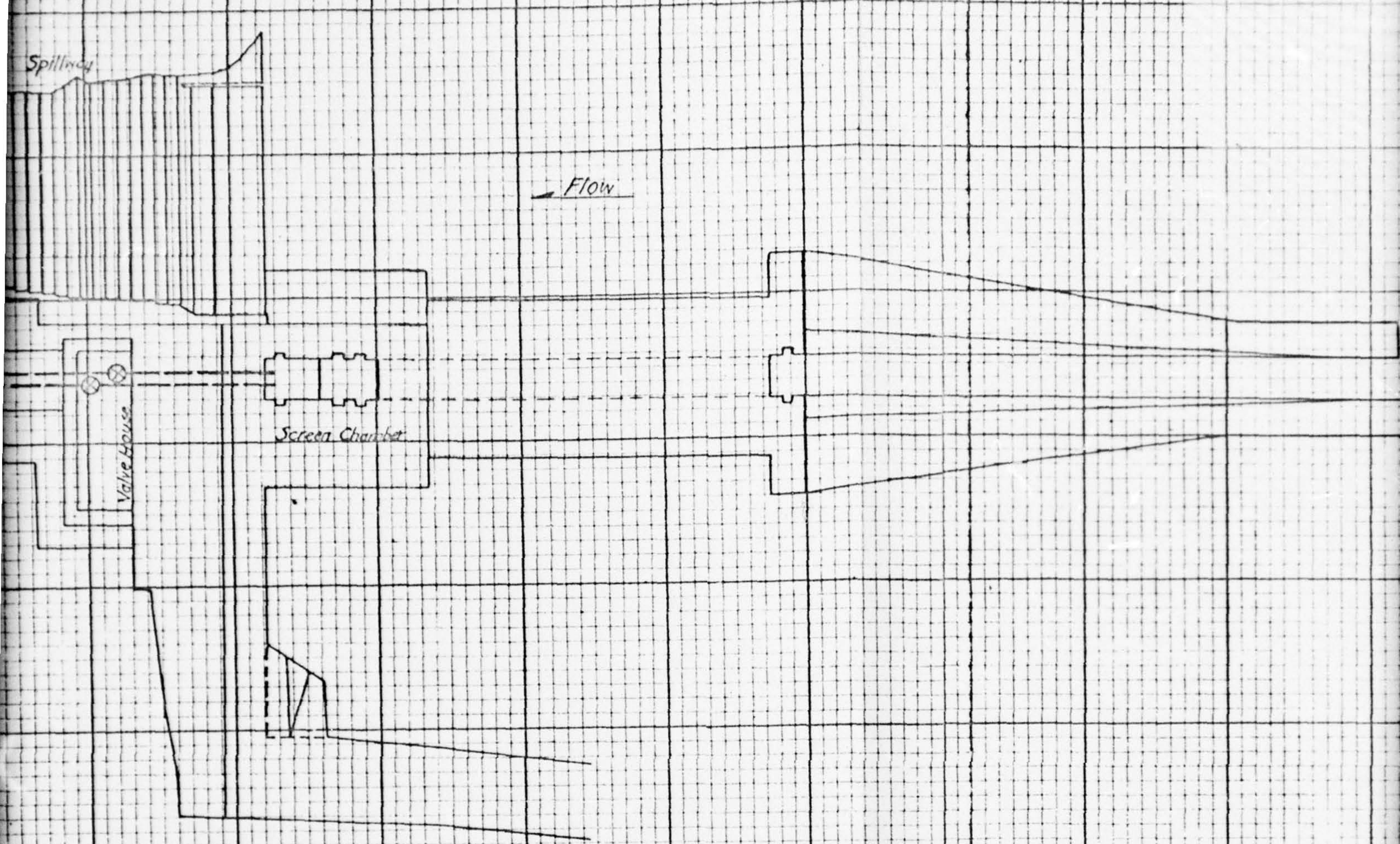
SECTIONS
SPILLWAY IMPROVEMENTS (1958)

MAY 1978 PLATE 6

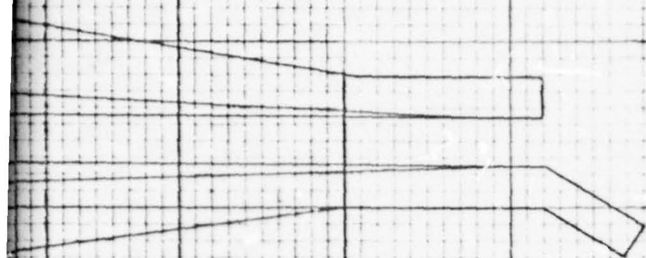
1



ELMHURST DAM
PLAN SHOWING INTAKE TUNNEL,
SCREEN CHAMBER & VALVE HOUSES.
Scale 1 inch = 20 feet


Spillway
Flow
Screen Chamber
Valve House

S.G.&W.Co.
Supply Reservoirs.
Sheet.



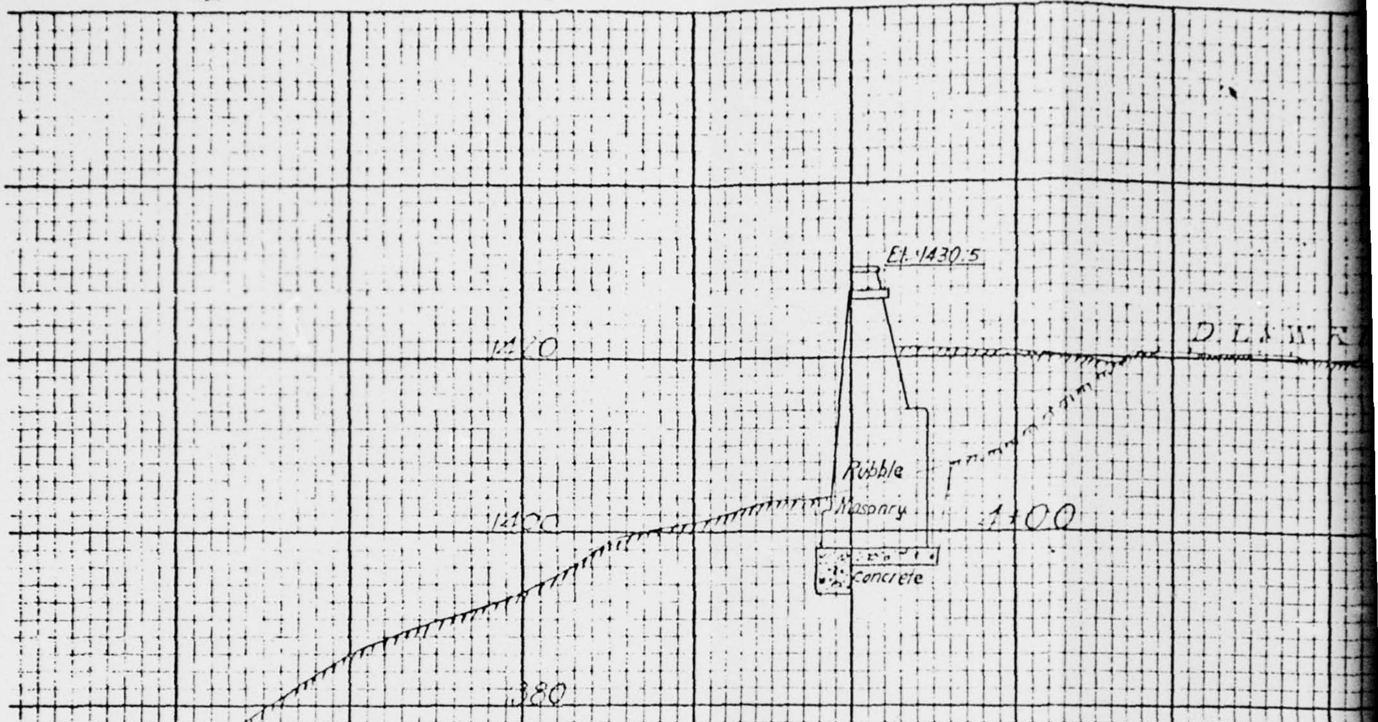
PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

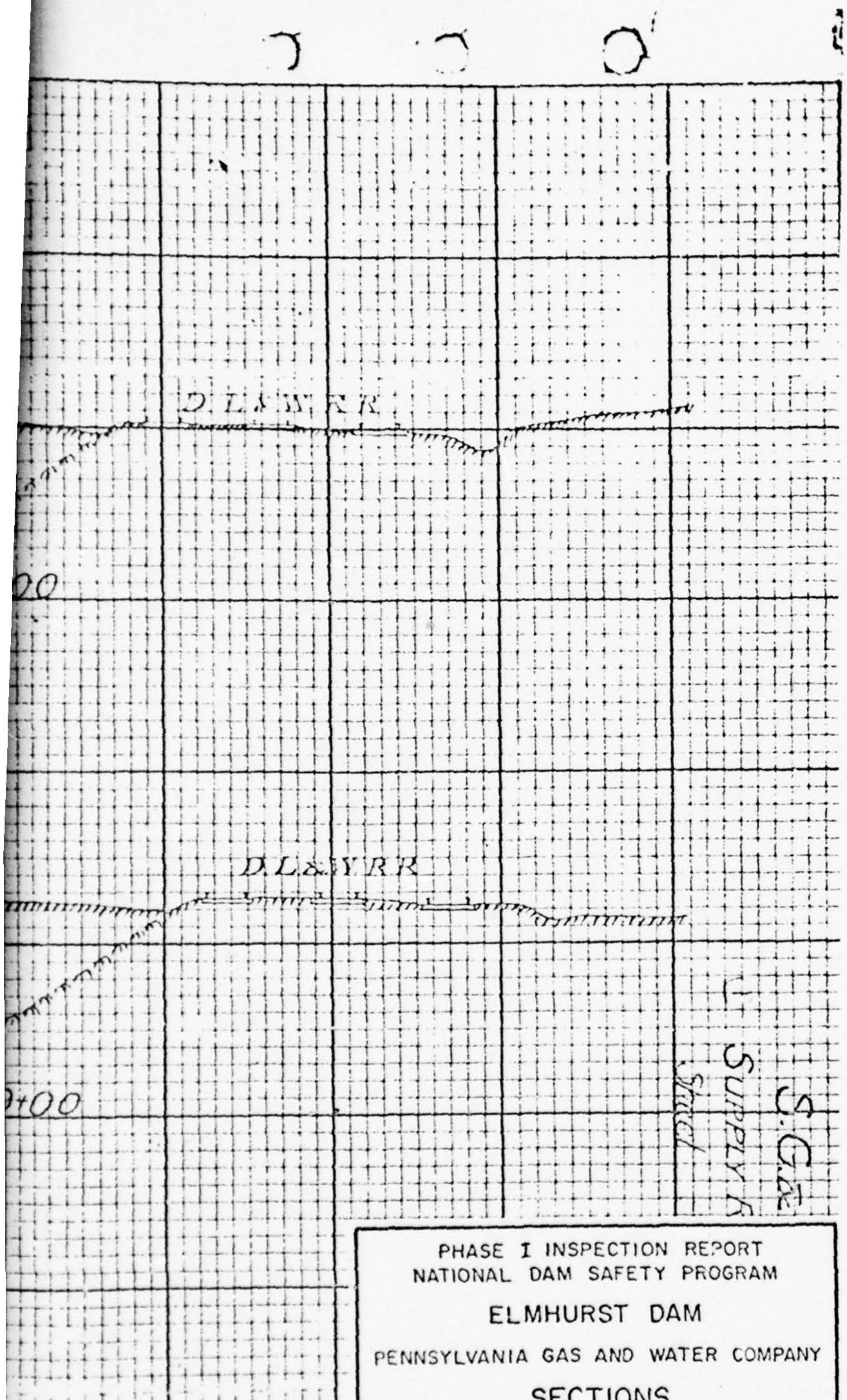
ELMHURST DAM

PENNSYLVANIA GAS AND WATER COMPANY

PLAN-OUTLET WORKS

MAY 1978 PLATE 7


El. 1430.5
1420
Rubble
Masonry
1400
Concrete
380
El. 1430.5
1420
Rubble
Masonry
1400
Concrete
9+00
ELMHURST DAM
SECTIONS THROUGH WALL
ALONG D. L. & W. R. R.
Scale 1 inch = 20 feet



PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

ELMHURST DAM

PENNSYLVANIA GAS AND WATER COMPANY

SECTIONS
LEFT ABUTMENT WALL

MAY 1978 PLATE 8

SUSQUEHANNA RIVER BASIN

ROARING BROOK, LACKAWANNA COUNTY

PENNSYLVANIA

ELMHURST DAM

PENNSYLVANIA GAS AND WATER COMPANY

PHASE I INSPECTION REPORT

NATIONAL DAM INSPECTION PROGRAM

MAY 1978

APPENDIX A

CHECKLIST - ENGINEERING DATA

CHECKLIST

ENGINEERING DATA

DESIGN, CONSTRUCTION, AND OPERATION
PHASE I

NAME OF DAM: Elmhurst

NDS ID NO.: 296 DER ID NO.: 35-18

ITEM	REMARKS
AS-BUILT DRAWINGS	Construction drawings of original structures and subsequent modifications.
REGIONAL VICINITY MAP	Project is shown on Moscow, Pennsylvania, Quadrangle Sheet N4115-W7530 17.5, 1946, Photo revised 1969
CONSTRUCTION HISTORY	Constructed 1889 by Scranton Gas and Water Company. Modified 1899, 1902, 1916, 1958
TYPICAL SECTIONS OF DAM	Available.
OUTLETS: Plan Details Constraints Discharge Ratings	Plans and sections available. No discharge ratings.

ENGINEERING DATA

ITEM	REMARKS
RAINFALL/RESERVOIR RECORDS	Hydrograph for August 1955 flood (flood of record).
DESIGN REPORTS	Design report for 1958 spillway addition; includes photos of model test.
GEOLOGY REPORTS	General geologic information on original drawings; boring logs for six holes for proposed 1958 modifications.
DESIGN COMPUTATIONS: Hydrology and Hydraulics Dam Stability Seepage Studies	1914 hydraulic and stability analysis of masonry spillway; 1958 stability analysis of concrete spillway.
MATERIALS INVESTIGATIONS: Boring Records Laboratory Field	Logs and blow counts for six borings made in 1958. No lab test records.
POSTCONSTRUCTION SURVEYS OF DAM	None

ENGINEERING DATA

ITEM	REMARKS
BORROW SOURCES	Materials obtained from onsite.
MONITORING SYSTEMS	Dam operators visit dam daily to check equipment and note water level.
MODIFICATIONS	1899: Spillway raised 3 feet, embankment raised 1 foot, core wall raised to within 6 inches of top embankment. 1902: Right sidewall raised 3 feet and extended; left sidewall raised 3 feet and extended; (Cont'd. on Sheet 4a)
HIGH POOL RECORDS	1955: 0.5 foot over left sidewall (0.5 foot from present maximum pool elevation).
POSTCONSTRUCTION ENGINEERING STUDIES AND REPORTS	1914 evaluation of hydraulics and stability for masonry spillway.
PRIOR ACCIDENTS OR FAILURE OF DAM: Description Reports	August 1955: Overtopping of left sidewall and damage to masonry spillway apron.

ENGINEERING DATA

ITEM	REMARKS
MAINTENANCE AND OPERATION RECORDS	No detailed operation records.
SPILLWAY: Plan Sections Details	Plans and sections of both spillways.
OPERATING EQUIPMENT: Plans Details	Plans and details available.
PREVIOUS INSPECTIONS Dates Deficiencies	1921: Slopes uneven; slight seepage. 1925: Seepage right abutment spillway; crack in left sidewall at alignment change. 1928: Seepage at toe retaining wall and through retaining wall. 1930: Erosion below apron. 1933: Seepage right abutment spillway. 1941: Damage at spillway apron. 1943: Deterioration of right wall along outlet channel. 1945: Seepage through masonry at right abutment and downstream face near gatehouse. 1953: No deficiencies noted. 1957: No deficiencies noted. 1965: No deficiencies noted.

ENGINEERING DATA

ITEM	REMARKS
MODIFICATIONS (Cont'd. from Sheet 3)	1902 (Cont'd.): 3-foot parapet wall added left of spillway. 1916: Lowered spillway by 3 feet and reinforced upper section with steel bars. 1958: Repairs from 1955 flood and built concrete spillway 136 feet long.

CHECKLIST

ENGINEERING DATA

HYDROLOGY AND HYDRAULICS

NAME OF DAM: Elmhurst NDS ID NO.: 296 DER ID NO.: 35-18

ELEVATION TOP NORMAL POOL (STORAGE CAPACITY): Elevation 1422.5

ELEVATION TOP FLOOD CONTROL POOL (STORAGE CAPACITY): Elevation 1431.5

ELEVATION MAXIMUM DESIGN POOL: Elevation 1431.5

ELEVATION TOP DAM: Elevation 1431.5

SPILLWAY CREST:
- a. Elevation Masonry: Elevation 1422.5 Concrete: Elevation 1422.5
- b. Type Masonry: Broad crested Concrete: Rounded crest
- c. Width Masonry: 9.5 feet Concrete: Not Applicable
- d. Length Masonry: 153.3 feet Concrete: 136 feet
- e. Location Spillover Masonry: Left Concrete: Right
- f. Number and Type of Gates None

OUTLET WORKS:
- a. Type Low level tunnel and cast-iron pipes
- b. Location Left abutment
- c. Entrance Inverts Tunnel: 1378.5; 48-inch CIP: 1380.0; 36-inch CIP: 1385.5.
- d. Exit Inverts Tunnel: not applicable; 48-inch CIP: 1378.6; 36-inch CIP: 1378.1.
- e. Emergency Draindown Facilities 36-inch and 48-inch cast-iron pipe

HYDROMETEOROLOGICAL GAGES:
- a. Type None
- b. Location None
- c. Records None

MAXIMUM NONDAMAGING DISCHARGE: Unknown.

SUSQUEHANNA RIVER BASIN

ROARING BROOK, LACKAWANNA COUNTY

PENNSYLVANIA

ELMHURST DAM

PENNSYLVANIA GAS AND WATER COMPANY

PHASE I INSPECTION REPORT

NATIONAL DAM INSPECTION PROGRAM

MAY 1978

APPENDIX B

CHECKLIST - VISUAL INSPECTION

CHECKLIST

VISUAL INSPECTION

PHASE I

Name of Dam: Elmhurst County: Lackawanna State: Pennsylvania

NDS ID No.: 296 DER ID No.: 35-18

Type of Dam: Earth with core wall Hazard Category: High

Date(s) Inspection: 4/10/78 and 4/11/78 Weather: Clear Temperature: 65° F.

General soil condition - moist

Pool Elevation at Time of Inspection: 1422.7 msl/Tailwater at Time of Inspection: 1372.0 msl

Inspection Personnel:

D. Wilson (GFCC)	D. Ebersole (GFCC)	
W. Seip (GFCC)	D. Kaufman (PG & W)	
J. Crouse (GFCC)		

D. Wilson (GFCC) Recorder

EMBANKMENT

Sheet 1 of 2

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SURFACE CRACKS	None	
UNUSUAL MOVEMENT OR CRACKING AT OR BEYOND THE TOE	None	
SLOUGHING OR EROSION: Embankment Slopes Abutment Slopes	5-foot diameter eroded area at toe adjacent to concrete spillway; 5-foot diameter low area 5 feet behind middle of second spillway monolith from bottom.	Eroded area has been filled with stone.
CREST ALIGNMENT: Vertical Horizontal	No abnormalities.	
RIPRAP FAILURES	None	

EMBANKMENT

Sheet 2 of 2

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
JUNCTION OF EMBANKMENT WITH: Abutment Spillway Other Features	No abnormalities.	
ANY NOTICEABLE SEEPAGE	None	
STAFF GAGE AND RECORDER	None	
DRAINS	Concrete spillway wall drain discharges into manhole near toe of embankment. Slight clear discharge.	Drain outlet had accumulation of clayey material.

CONCRETE/MASONRY DAMS
(Masonry Spillway and Gravity Section)
Sheet 1 of 2

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
ANY NOTICEABLE SEEPAGE	Downstream face of spillway - too much discharge to inspect spillway left wall behind 48-inch diameter blowoff - leakage at base of spillway left wall downstream of terminal wall - 4-5 gpm near base.	Gravity section downstream face - leakage at two points above gatehouse.
JUNCTION OF STRUCTURE WITH: Abutment Embankment Other Features	18-inch diameter sinkhole at junction of spillway left wall and gatehouse just above 48-inch diameter blowoff. Slight erosion of backfill at left side of gatehouse.	
DRAINS	Spillway right wall: 4-inch diameter at downstream end; slight weeping. Spillway left wall: 4-inch diameter at downstream end - dry.	
WATER PASSAGES	None	
FOUNDATION	No evidence of foundation problems.	

CONCRETE/MASONRY DAMS
(Masonry Spillway and Gravity Section)
Sheet 2 of 2

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONCRETE SURFACES: Surface Cracks Spalling	Slight spalling of concrete on spillway left wall near downstream end.	Concrete is a repaired section of masonry wall.
STRUCTURAL CRACKING	Two wide vertical cracks near top spillway left wall near downstream end. Cracks extend through wall.	Cracks are in concrete at repaired section of masonry wall. No movement at cracks.
ALIGNMENT: Vertical Horizontal	No irregularities.	
MONOLITH JOINTS	Joint filler missing between two apron slabs at right side spillway just below cascade.	All other joints filled.
CONSTRUCTION JOINTS	Mortar joints are generally sound.	
STAFF GAGE OR RECORDER	None	

OUTLET WORKS

Sheet 1 of 1

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CRACKING AND SPALLING OF CONCRETE SURFACES IN OUTLET CONDUIT	Upstream masonry tunnel inaccessible.	Bulkheads available in screen chamber for dewatering, but PG&W Co. did not want 36-inch supply line temporarily taken out of service.
INTAKE STRUCTURE	Masonry above water line appeared satisfactory.	
OUTLET STRUCTURE Valve Chambers	Water 3 inches deep on valve chamber floors.	
OUTLET CHANNEL	None.	
EMERGENCY GATE	Four men opened 48-inch discharge valve 6 inches in 40 minutes. The 36-inch valve was not opened, but appeared lubricated.	PG&W Co. concerned that farther opening of 48-inch valve would draw sediment into 36-inch water supply line.

UNGATED SPILLWAY
(Concrete Spillway)
Sheet 1 of 1

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONCRETE WEIR	Exhibits normal wear; all joints sealed.	
APPROACH CHANNEL	Clear; no operating constraints.	
DISCHARGE CHANNEL	Excellent condition.	
BRIDGE AND PIERS	Pier between concrete and masonry spillways.	No deficiencies noted.
SPILLWAY RIGHT WALL	Signs of possible distress in second and third monolith from downstream end: 3 fine diagonal cracks; wide (1/8 inch) monolith separation; joint spalling back face at monolith joint.	5-inch diameter hole at joint at base of second and third monoliths; possible differential movement indicated by monolith misalignment at top wall.

INSTRUMENTATION

Sheet 1 of 1

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
MONUMENTATION/SURVEYS	None	
OBSERVATION WELLS	None	
WEIRS	None	
PIEZOMETERS	None	
OTHER	None	

RESERVOIR AND WATERSHED

Sheet 1 of 1

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SLOPES	Generally mild slopes; no evidence of instability.	
SEDIMENTATION	No significant problem reported by owner.	
WATERSHED DESCRIPTION	Wooded and partially developed.	

DOWNSTREAM CHANNEL

Sheet 1 of 1

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONDITION: Obstructions Debris Other	Growth of small brush (signs of complete cutting previous year).	
SLOPES	No evidence of instability or erosion.	
APPROXIMATE NUMBER OF HOMES AND POPULATION	Communities of Dunmore, Elmhurst and Scranton.	

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
RIGHT SIDEWALL	Top 6 inches of concrete coping cracked and heaved near upstream end; some standing water observed behind wall.	Would allow some discharge around right side of dam if pool level were within 6 inches of top of dam.
LEFT SIDEWALL	Disintegration of concrete coping at eight locations to maximum depth of 6 inches. About 80 L.F. affected.	Would allow some discharge around left side of dam if pool level were within 6 inches of top of dam.
	Seepage through back face in first 300 feet upstream from dam about 3.5 feet above backfill.	Seepage is worst near dam; particularly within 100 feet of dam.
	Wide diagonal crack at offset in wall.	No movement noted at diagonal crack.
	Mortar on back face missing and deteriorating at top 3 feet.	
	Wall drain collects seepage and discharges into outlet channel at gatehouse.	Drain pipe is on surface for last 25 feet. Poor joint condition allows some flow escape from pipe.

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SURFACE RUNOFF SYSTEM ALONG TOE EMBANKMENT	Poor pipe joint has caused 18-inch diameter sinkhole. Some flow from ditch flows under a pipe that is supposed to receive it. Erosion of 5-foot wide by 12-foot long by 4-foot maximum depth area at end of spillway right wall. Evidence of uncontrolled overland flow over top of first monolith from downstream end of spillway right wall. Has caused some minor erosion damage along toe embankment. The source of an 8-inch diameter pipe that carried most of flow into manhole could not be located.	

SUSQUEHANNA RIVER BASIN

ROARING BROOK, LACKAWANNA COUNTY

PENNSYLVANIA

ELMHURST DAM

PENNSYLVANIA GAS AND WATER COMPANY

PHASE I INSPECTION REPORT

NATIONAL DAM INSPECTION PROGRAM

MAY 1978

APPENDIX C

HYDROLOGY AND HYDRAULICS

CLASSIFICATION

HIGH HAZARD, SINCE POPULATION DOWNSTREAM IS 1,000

INTERMEDIATE SIZE, SINCE HEIGHT = 68 FEET AND CAPACITY = 1,220 MILLION GALLONS
REFERENCE: "RECOMMENDED GUIDELINES FOR SAFETY INSPECTION OF DAMS," p. D-8.

SPILLWAY DESIGN FLOOD (SDF)

THE SDF SHOULD BE THE PMF (FROM p. D-12 OF "REC. GUIDELINES...")

HYDROLOGY AND HYDRAULICS ANALYSIS
REFERENCE: PHASE I PROCEDURE PACKAGE

II. A. 2. PMF INFLOW HYDROGRAPH NOT AVAILABLE

a. FROM CONVERSATION OF MIKE KAYOWITZ WITH ACH, PRORATE PMF AT ELMHURST, D.A. = 37.3 SQ. MI., WITH PMF PEAK OF 39,500 CFS AT STILLWATER, D.A. = 36.9 SQ. MI.

GENERALIZED FORM OF TRANSPOSITION (REFERENCE: BALTIMORE CONTACT - MIKE KAYOWITZ)

$$\frac{Q_1}{Q_2} = \left(\frac{D.A._1}{D.A._2}\right)^{0.8}$$

$$\text{OR, } Q_1 = Q_2\left(D.A._1 / D.A._2\right)^{0.8}$$

$$Q_1 = 39{,}500\left(37.3/36.9\right)^{0.8}$$

$$Q_1 = 39{,}930 \text{ CFS}$$

$\therefore X = 39{,}930$ CFS

= PMF PEAK INFLOW FROM STORM OVER ENTIRE ELMHURST WATERSHED

EFFECT OF UPSTREAM RESERVOIRS

NEGLECT EFFECTS OF LAKE HENRY AND HOLLISTER RESERVOIRS

DO NOT NEGLECT EFFECT OF CURTIS RESERVOIR

PROPORTION OF ELMHURST PMF PEAK AT CURTIS RESERVOIR

$$\frac{\text{CURTIS COMPONENT PMF PEAK}}{\text{CURTIS DRAINAGE AREA}} = \frac{\text{TOTAL PMF PEAK AT ELMHURST}}{\text{TOTAL D.A. AT ELMHURST}}$$

8313

GANNETT FLEMING CORDDRY AND CARPENTER, INC. HARRISBURG, PA.	SUBJECT: ELMHURST DAM (35-13) HYDROLOGY AND HYDRAULICS ANALYSIS	FILE NO. 7613 'A' SHEET NO. 2 OF 12 SHEETS
	FOR: U.S.C.E. - BALTIMORE DISTRICT	
	COMPUTED BY: JMC DATE: 4/23/78	CHECKED BY: DAW DATE: 5/5

$$\frac{Y}{2.4 \text{ SQ. MI.}} = \frac{39{,}930 \text{ CFS}}{37.3 \text{ SQ. MI.}}$$

$\therefore Y = 2{,}569$ CFS

PMF PEAK INFLOW FROM STORM OVER CURTIS WATERSHED

II. A. 2. PMF INFLOW HYDROGRAPH NOT AVAILABLE

a. PRORATE PMF AT CURTIS, D.A. = 2.4 SQ. MI., WITH PMF PEAK OF 9,700 CFS AT FALLING BROOK, D.A. = 4.14 SQ. MI.

GENERALIZED FORM OF TRANSPOSITION (REFERENCE: BALTIMORE CONTACT - MIKE KANUNITE)

$$\frac{Q_1}{Q_2} = \left(\frac{D.A._1}{D.A._2}\right)^{0.8}$$

$$\text{OR,} \quad Q_1 = Q_2\left(D.A._1 / D.A._2\right)^{0.8}$$

$$Q_1 = 9{,}700\left(2.4/4.14\right)^{0.8}$$

$$Q_1 = 6{,}270 \text{ CFS}$$

$$\therefore Q_1 = 6{,}270 \text{ CFS}$$

APPROACH BY 2 CASES AND CHECK SPILLWAY ADEQUACY FOR:

CASE 1 - STORM OVER ENTIRE ELMHURST WATERSHED
ROUTE CURTIS COMPONENT THROUGH CURTIS RESERVOIR AND ADD CONTRIBUTION FROM REST OF WATERSHED.

CASE 2 - STORM OVER CURTIS WATERSHED ALONE
ROUTE CURTIS PMF THROUGH CURTIS RESERVOIR. IF CURTIS DAM SHOULD FAIL, LOAD CURTIS VOLUME ON NORMAL POOL ELEVATION AT ELMHURST AND CHECK FOR OVERTOPPING OF ELMHURST DAM.

ADDITIONAL DATA REQUIREMENTS AT CURTIS: SPILLWAY CAPACITY
STORAGE ABOVE NORMAL POOL

8313

GANNETT FLEMING CORDDRY AND CARPENTER, INC.
HARRISBURG, PA.

SUBJECT: ELMHURST DAM (35-13) — HYDROLOGY AND HYDRAULICS ANALYSIS   FILE NO. 7613.1A   SHEET NO. 3 OF 12 SHEETS
FOR: USCE - BALTIMORE DISTRICT
COMPUTED BY: JMC   DATE: 4/29/79   CHECKED BY: DAW   DATE: 5/5

CASE 1 - STORM OVER ENTIRE ELMHURST WATERSHED

INFLOW TO ELMHURST RESERVOIR OTHER THAN CURTIS RESERVOIR CONTRIBUTION
= ADJUSTED PMF PEAK AT ELMHURST - CURTIS COMPONENT OF PMF PEAK
= 39,930 - 2,569 = 37,361 CFS = 37,360 CFS FOR PMF (ROUNDED)
AND FOR ½ PMF PEAK = 37,360 / 2 = 18,680 CFS

INFLOW TO ELMHURST RESERVOIR FROM CURTIS RESERVOIR
a) INFLOW TO CURTIS RESERVOIR = 2,569 CFS FOR PMF PEAK
= 2,569/2 = 1,285 CFS (ROUNDED) FOR ½ PMF PEAK

b) OUTFLOW FROM CURTIS RESERVOIR ≈ INFLOW TO ELMHURST RESERVOIR
CURTIS DAM DATA (TELCON DEW/ DK - 5/1/79)

TOP OF DAM ELEVATION	1499.6'
MAIN SPILLWAY ELEVATION	1495.8'
AUXILIARY SPILLWAY ELEVATION	1496.3'
MAIN SPILLWAY LENGTH	52.4'
AUXILIARY SPILLWAY LENGTH	57.5'
HEIGHT OF DAM	< 40'
RESERVOIR AREA AT SPILLWAY CREST	75.14 ACRES
RESERVOIR VOLUME AT SPILLWAY CREST	418,333 MG

ASSUME THAT BOTH SPILLWAYS ARE BROAD-CRESTED WEIRS
BROAD-CRESTED WEIR EQUATION (p 372 VENNARD, ELEMENTARY FLUID MECHANICS)

$$q = \sqrt{g(2H/3)^3}$$

$$\therefore Q = L\sqrt{g(2H/3)^3}$$

WEIR EQUATION

ELEV	H_M	H_A	MAIN SPILLWAY $Q = 52.4\sqrt{g(2H/3)^3}$	AUXILIARY SPILLWAY $Q = 57.5\sqrt{g(2H/3)^3}$	TOTAL Q (CFS)
1496.3	0.5	0.0	57	0	57
1496.8	1.0	0.5	162	63	225
1497.3	1.5	1.0	297	178	475
1497.8	2.0	1.5	458	326	784
1498.3	2.5	2.0	640	502	1,142
1498.8	3.0	2.5	841	702	1,543
1499.3	3.5	3.0	1,060	923	1,983
1499.6	3.8	3.3	1,199	1,065	2,264

8313

GANNETT FLEMING CORDDRY AND CARPENTER, INC.
HARRISBURG, PA.

SUBJECT: ELMHURST DAM (35-13) FILE NO. 7613.1A
HYDROLOGY AND HYDRAULICS ANALYSIS SHEET NO. 4 OF 12 SHEETS
FOR: USCE - BALTIMORE DISTRICT
COMPUTED BY: JMC DATE: 5/1/78 CHECKED BY: DAW DATE: 5/5

∴ SPILLWAY CAPACITY BEFORE OVERTOPPING = 2,264 CFS (CURTIS)

II. B. ABILITY OF CURTIS SPILLWAY TO PASS CURTIS COMPONENT OF ELMHURST PMF PEAK

1. CAPACITY OF CURTIS SPILLWAY = 2,264 CFS

3. THE CURTIS COMPONENT OF THE ELMHURST PMF PEAK FLOW IS GREATER THAN THE SPILLWAY CAPACITY (2,569 > 2,264)

b. ROUTING IS NOT AVAILABLE

(1.) THE SPILLWAY WILL PASS (2,264/2,569) = 0.881 = 88.1% = p OF THE PEAK

(2) INCLUDES 3 METHOD TO ESTIMATE STORAGE EFFECT OF RESERVOIR

(a) TRIANGLE SHAPE FOR PMF HYDROGRAPH

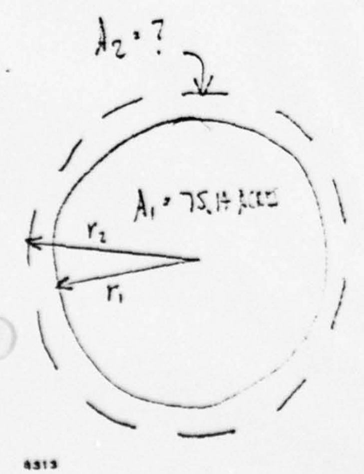
(b) BASE TIME COMPUTATION — CURTIS TIME < TOTAL TIME (T) < ELMHURST TIME

T = T FOR CURTIS DAM = 22 HOURS	T = T FOR ELMHURST DAM = 41.6 HOURS
$1-p = 1.0 - 0.881 = 0.119 = \frac{\Delta AOC}{\Delta AOB}$	$1-p = 1.0 - 0.881 = 0.119 = \frac{\Delta AOC}{\Delta AOB}$
$\Delta AOB = \frac{1}{2}bh = \frac{1}{2}(22)(2,569) = 28,260$ CFS-HR	$\Delta AOB = \frac{1}{2}bh = \frac{1}{2}(41.6)(2,569) = 53,435$ CFS-HR
$\Delta AOC = (1-p)\,\Delta AOB = (0.119)\,28,260 = 3,363$ CFS-HR	$\Delta AOC = (1-p)\,\Delta AOB = (0.119)(53,435) = 6,359$ CFS-HR
3,363 CFS-HR OF STORAGE IS REQUIRED	6,359 CFS-HR OF STORAGE IS REQUIRED
$3,363 \text{ CFS-HR} \times \frac{3600 \text{ AC-SEC}}{43,560 \text{ FT}^2\text{-HR}} = 278$ AC-FT	$6,359 \text{ CFS-HR} \times \frac{3600 \text{ AC-SEC}}{43,560 \text{ FT}^2\text{-HR}} = 526$ AC-FT

278 AC-FT < REQUIRED STORAGE < 526 AC-FT

(c) INCREMENTAL STORAGE AVAILABLE BETWEEN NORMAL POOL ELEVATION AND MAXIMUM POOL ELEVATION

NORMAL POOL ELEVATION = SPILLWAY CREST ELEVATION = 1435.8'
MAXIMUM POOL ELEVATION = TOP OF DAM ELEVATION = 1439.6'
AREA OF RESERVOIR WITH W.S. @ SPILLWAY CREST = 75.14 ACRES
AREA OF RESERVOIR WITH W.S. @ TOP OF DAM = ?
ASSUME RESERVOIR SIDE SLOPES OF 2H ON 1V AND ASSUME CIRCULAR SHAPE
∴ WITH 2H ON 1V SLOPE AND $\Delta V = 3.8'$, $\Delta H = 2(3.8') = 7.6'$



$A = \pi r^2$, WHERE r = EQUIVALENT RADIUS OF ASSUMED CIRCULAR SHAPE

$75.14 \text{ ACRES} \times \frac{43,560 \text{ FT}^2}{1 \text{ ACRE}} = \pi r_1^2$

$1,043,000 = r^2$

$r_1 = 1,020.7$ FEET

$r_2 = r_1 + 7.6' = 1,020.7 + 7.6 = 1,028.3'$

$A_2 = \pi r_2^2 = \pi(1,028.3)^2 = 3,322,000 \text{ FT}^2$

$A_2 = 76.26$ ACRES

INCREMENTAL STORAGE = $\left(\frac{A_1 + A_2}{2}\right) \times \Delta V$

$= \left(\frac{75.44 + 72.25}{2}\right) 3.9' = 287.7$ AC-FT

INCREMENTAL STORAGE = 288 AC-FT

278 AC-FT < 288 AC-FT < 526 AC-FT

OUTFLOW FROM CURTIS RESERVOIR INTO ELMHURST RESERVOIR

SINCE CURTIS RESERVOIR IS NOT A FLOOD CONTROL DAM, THE RESERVOIR AND SPILLWAYS HAVE VERY LITTLE ATTENUATING EFFECT ON FLOOD FLOWS INTO THE RESERVOIR. HOWEVER, ASSUMING ZERO OUTFLOW FROM CURTIS IS CONSERVATIVE. THAT IS, IF THE ELMHURST SPILLWAY CANNOT PASS THE PMF WITHOUT CONSIDERING THE CURTIS CONTRIBUTION, THE ELMHURST SPILLWAY CANNOT PASS THE TOTAL INFLOW INTO ELMHURST RESERVOIR. CHECK PMF INFLOW WITHOUT CONSIDERING THE OUTFLOW FROM CURTIS DAM.

∴ MINIMUM TOTAL INFLOW INTO ELMHURST RESERVOIR FOR PMF INFLOW = 37,360 CFS

II. B. ABILITY OF ELMHURST SPILLWAY TO PASS ELMHURST PMF PEAK

1. CAPACITY OF ELMHURST SPILLWAY = 31,000 CFS (1958 MODEL STUDY)
3. THE PMF PEAK FLOW IS GREATER THAN THE SPILLWAY CAPACITY (37,360 > 31,000)
   b. ROUTING OF THE PMF IS NOT AVAILABLE
      (1) THE SPILLWAY WILL PASS (31,000/37,360) = 0.830 = 83.0% = p OF THE PMF PEAK
      (2.) INCLOSURE 3 METHOD TO ESTIMATE STORAGE EFFECT OF RESERVOIR
         (a) TRIANGULAR SHAPE FOR PMF HYDROGRAPH
         (b) FROM GRAPH OF TOTAL TIME VS. D.A. FOR SUSQUEHANNA RIVER BASIN,

TOTAL TIME, T = 41.6 HOURS

$1 - p = 1.0 - 0.830 = 0.170 = \frac{\Delta AOC}{\Delta AOB}$

$\Delta AOB = \frac{1}{2} bh = \frac{1}{2}(41.6 \text{ HOURS})(37,360 \text{ CFS}) = 777,100$ CFS-HOURS

SUBSTITUTING, $\Delta AOC = (0.170)\,\Delta AOB = (0.170)(777,100) = 132,100$ CFS-HOURS

∴ 132,100 CFS-HOURS OF STORAGE IS REQUIRED TO PASS PMF WITHOUT OVERTOPPING

$132,100 \frac{FT^3}{SEC} \times \text{HOURS} \times \frac{60 \text{ MIN}}{1 \text{ HOUR}} \times \frac{60 \text{ SEC}}{1 \text{ MIN}} \times \frac{1 \text{ ACRE}}{43,560 \text{ FT}^2} = 10,920$ AC-FT (MINIMAL)

(c) INCREMENTAL STORAGE AVAILABLE BETWEEN NORMAL POOL ELEVATION AND MAXIMUM POOL ELEVATION

NORMAL POOL ELEVATION = SPILLWAY CREST ELEVATION = 1422.5'

MAXIMUM POOL ELEVATION = TOP OF DAM ELEVATION = 1431.5'

8313

GANNETT FLEMING CORDDRY
AND CARPENTER, INC.
HARRISBURG, PA.

SUBJECT: ELMHURST DAM (35-13) FILE NO. 7613.1A
HYDROLOGY AND HYDRAULICS ANALYSIS SHEET NO. 6 OF 12 SHEETS
FOR: USCE- BALTIMORE DISTRICT
COMPUTED BY: JMC DATE: 5/1/79 CHECKED BY: DAW DATE: 5/5

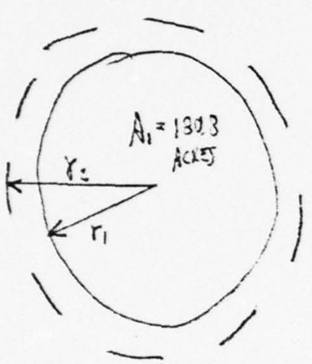
AREA OF RESERVOIR WITH W.S. AT SPILLWAY CREST = 180.8 ACRES
(FROM 1958 WITKIN REPORT)

AREA OF RESERVOIR WITH W.S. @ TOP OF DAM = ?

ASSUME RESERVOIR SIDE SLOPES OF 2H ON 1V AND ASSUME CIRCULAR SHAPE

∴ WITH 2H ON 1V SLOPE AND $\Delta V = 9.0'$, $\Delta H = 2(9.0') = 18.0'$

$A_2 = ?$



$A = \pi r^2$, WHERE r = EQUIVALENT RADIUS OF ASSUMED CIRCULAR SHAPE

$180.8 \text{ ACRES} \times \frac{43,560 \text{ FT}^2}{1 \text{ ACRE}} = \pi r_1^2$

$2,507,000 = r_1^2$

$r_1 = 1,583.3'$

$r_2 = r_1 + 18.0' = 1,583.3' + 18.0' = 1,601.3$

$A_2 = \pi r_2^2$

$A_2 = \pi (1,601.3')^2 = 8,056,000 \text{ FT}^2$

$A_2 = 184.9$ ACRES

INCREMENTAL STORAGE $= \left(\frac{A_1 + A_2}{2}\right) \times \Delta V$

$= \left(\frac{180.8 + 184.9}{2}\right) 9.0'$

$= 1,646$ AC-FT

STORAGE REQUIRED = 10,920 AC-FT > STORAGE AVAILABLE = 1,646 AC-FT

C. PROCEDURES FOR DETERMINATION OF ADEQUATE / INADEQUATE SPILLWAY CAPACITY

2. STORAGE REQUIRED IS GREATER THAN STORAGE AVAILABLE

a. ETL 1110-2- STATES THREE CONDITIONS THAT MUST EXIST BEFORE THE SPILLWAY CAPACITY IS CONSIDERED TO BE SERIOUSLY INADEQUATE. CONDITION "C." MAY OR MAY NOT BE MET (I.E. IS THE SPILLWAY ABLE TO PASS ½ PMF W/O OVERTOPPING FAILURE?)

b. REPEAT CALCULATIONS FOR ½ PMF PEAK

½ PMF INFLOW TO ELMHURST = ½ PMF INFLOW TO ELMHURST OTHER THAN CURTIS RESERVOIR CONTRIBUTION + OUTFLOW FROM CURTIS RESERVOIR FOR ½ PMF OVER ENTIRE ELMHURST AREA

$= \frac{1}{2}(37,360) + \text{MAX. INFLOW} = \frac{1}{2}(37,360) + (1.00)\frac{1}{2}(2,569)$

$= 19,965$ CFS

II. B. ABILITY OF ELMHURST SPILLWAY TO PASS ELMHURST ½ PMF PEAK

1. CAPACITY OF SPILLWAY = 31,000 CFS

2. ½ PMF FLOW IS LESS THAN THE SPILLWAY CAPACITY (19,965 < 31,000)

a. NO SPILLWAY ROUTING FOR ½ PMF IS NECESSARY

b. THE DAM CAN BE ASSUMED TO BE ABLE TO PASS ½ PMF WITHOUT OVERTOPPING

II. C. 2. c. TAILWATER AT INSTANT BEFORE OVERTOPPING OCCURS

SPILLWAY CAPACITY DISCHARGE = 28,700 CFS ; FROM HEC-2 COMPUTER RUN,

TAILWATER DEPTH @ Q = 28,700 CFS IS 15.6 FEET

TOP OF DAM ELEVATION = 1431.5'

HEIGHT OF DAM = 68'

BOTTOM OF DAM ELEV. = 1363.5'

TAILWATER DEPTH = 15.6'

TAILWATER ELEVATION = 1379.1'

TOP OF DAM ELEV. − TAILWATER ELEVATION = 1431.5 − 1379.1' = 52.4'

FROM GRAPH ON SHEET 8, TAILWATER DEPTH FOR Q = 31,000 CFS = 16.2' ∴ TAILWATER ELEVATION = 1379.7'

CASE 2 - STORM OVER CURTIS WATERSHED ALONE

II. B. ABILITY OF CURTIS SPILLWAY TO PASS CURTIS PMF

1. CAPACITY OF CURTIS SPILLWAY = 2,264 CFS

3. THE CURTIS PMF PEAK FLOW IS GREATER THAN THE SPILLWAY CAPACITY (6,270 > 2,264)

b. ROUTING OF THE PMF IS NOT AVAILABLE

(1) THE SPILLWAY WILL PASS (2,264/ 6,270) = 0.361 = 36.1% = p OF THE PMF PEAK

(2) INCLOSURE 3 METHOD TO ESTIMATE STORAGE EFFECT OF RESERVOIR

(a) TRIANGULAR SHAPE FOR PMF HYDROGRAPH

(b) FROM GRAPH OF TOTAL TIME VS. D.A. FOR SUSQUEHANNA RIVER BASIN,

TOTAL TIME, T = 22 HOURS

$1-p = 1.0 - 0.361 = 0.639 = \frac{\Delta AOC}{\Delta AOB}$

$\Delta AOB = \frac{1}{2} bh = \frac{1}{2}(22 \text{ HOURS})(6,270 \text{ CFS}) = 68,970$ CFS-HOURS

SUBSTITUTING, $\Delta AOC = (0.639)\,\Delta AOB = (0.639)(68,970) = 44,072$ CFS-HOURS

∴ 44,072 CFS-HOURS OF STORAGE IS REQUIRED TO PASS PMF WITHOUT OVERTOPPING

$\frac{FT^3}{SEC} \times \text{HOURS} \times \frac{60 \text{ SEC}}{1 \text{ MIN}} \times \frac{60 \text{ MIN}}{1 \text{ HOUR}} \times \frac{1 \text{ ACRE}}{43,560 \text{ FT}^2} = 3,642$ AC-FT REQUIRED

(c) INCREMENTAL STORAGE AVAILABLE BETWEEN NORMAL POOL ELEVATION AND MAXIMUM POOL ELEVATION (SEE SHEETS 4 AND 5) = 288 AC-FT

STORAGE REQUIRED = 3,642 AC-FT > STORAGE AVAILABLE = 288 AC-FT

II B. ABILITY OF CURTIS SPILLWAY TO PASS ½ CURTIS PMF

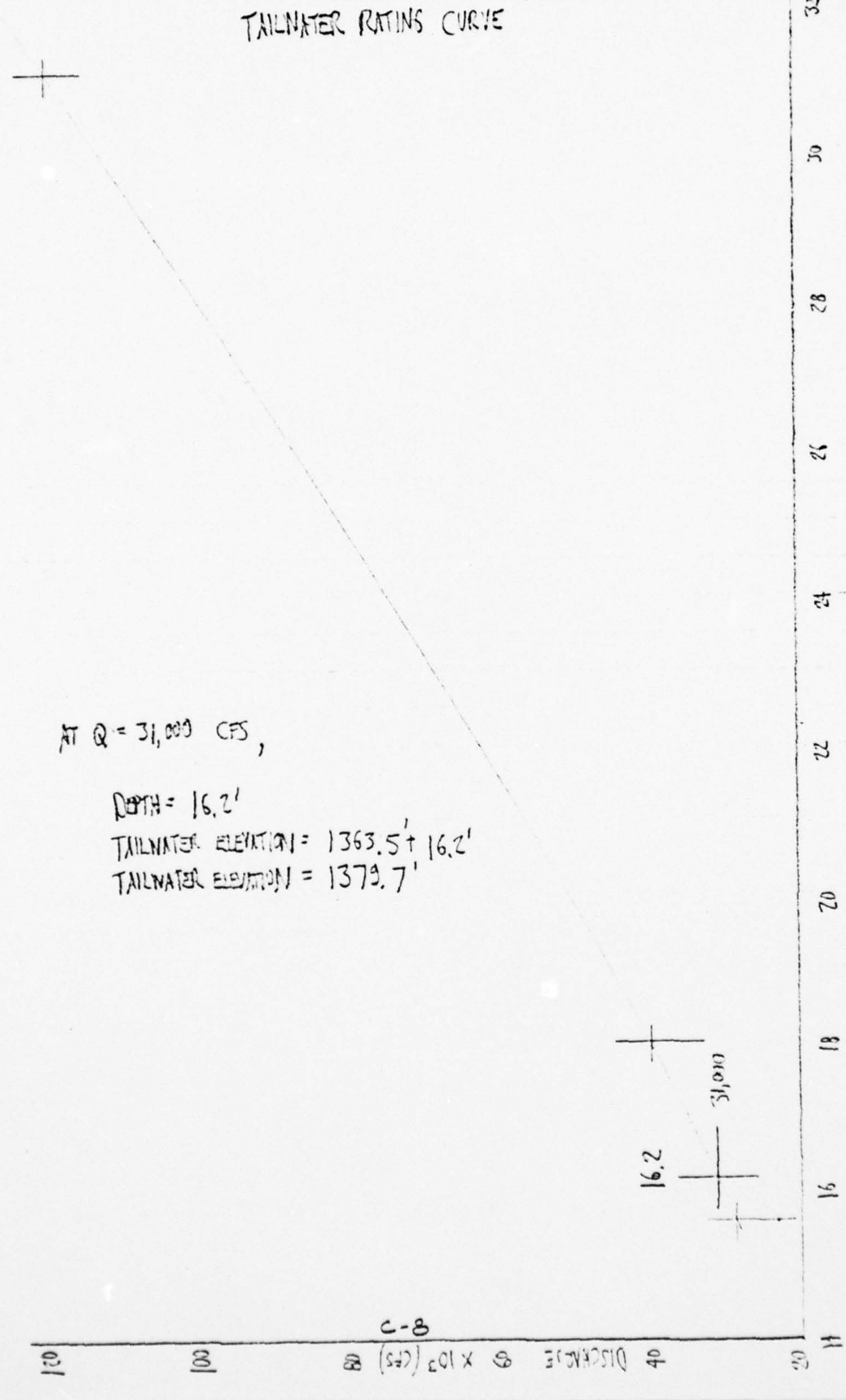
1. CAPACITY OF CURTIS SPILLWAY = 2,264 CFS

3. ½ CURTIS PMF PEAK FLOW IS GREATER THAN THE SPILLWAY CAPACITY (3,135 > 2,264)

GANNETT FLEMING CORDDRY AND CARPENTER, INC.
HARRISBURG, PA.

SUBJECT ELMHURST DAM (35-19) FILE NO. 7613.1A
HYDROLOGY AND HYDRAULICS SHEET NO. 8 OF 12 SHEETS
FOR USCE - BALTIMORE DISTRICT
COMPUTED BY JAC DATE 5/6/78 CHECKED BY DAN DATE 5/?

TAILWATER RATING CURVE



AT Q = 31,000 CFS,

DEPTH = 16.2'
TAILWATER ELEVATION = 1363.5' + 16.2'
TAILWATER ELEVATION = 1379.7'

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b. ROUTING OF $\frac{1}{2}$ PMF IS NOT AVAILABLE

(1) THE SPILLWAY WILL PASS $(2,264/3,135) = 0.722 = 72.2\% = p$ OF $\frac{1}{2}$ PMF PEAK

(2.) INCLUDE 3 METHOD TO ESTIMATE STORAGE EFFECT OF RESERVOIR

(a) TRIANGULAR SHAPE FOR $\frac{1}{2}$ PMF HYDROGRAPH

(b) SAME AS BEFORE, EXCEPT THAT PEAK IS NOW 3,135 CFS

$1 - p = 1.0 - 0.722 = 0.278 = \frac{\Delta ADC}{\Delta ADB}$

$\Delta ADB = \frac{1}{2} bh = \frac{1}{2}(22 \text{ HOURS})(3,135 \text{ CFS}) = 34,485$ CFS-HOURS

SUBSTITUTING, $\Delta ADC = (0.278)\ \Delta ADB = (0.278)(34,485) = 9,587$ CFS-HOURS

$\therefore$ 9,587 CFS-HOURS IS REQUIRED TO PASS $\frac{1}{2}$ PMF W/O OVERTOPPING

$9,587 \frac{FT^3}{SEC} \times \text{HOURS} \times \frac{60 \text{ MIN}}{1 \text{ HOUR}} \times \frac{60 \text{ SEC}}{1 \text{ MIN}} \times \frac{1 \text{ ACRE}}{43,560 \text{ FT}^2} = 792$ AC-FT REQUIRED

(c) INCREMENTAL STORAGE AVAILABLE BETWEEN NORMAL POOL ELEVATION AND MAXIMUM POOL ELEVATION - SEE SHEETS 4 & 5 - = 288 AC-FT

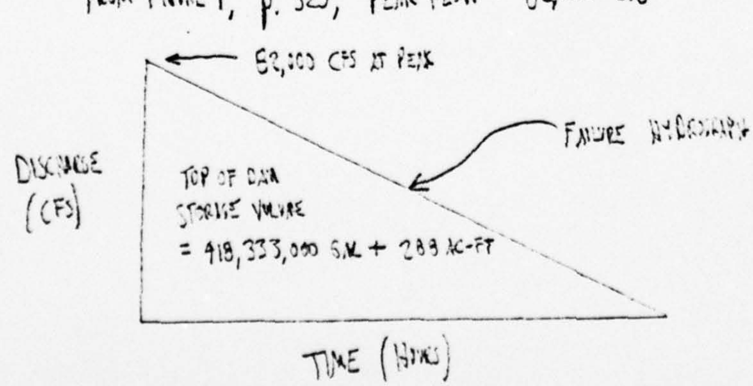
STORAGE REQUIRED = 792 AC-FT > STORAGE AVAILABLE = 288 AC-FT

$\therefore$ THE CURTIS SPILLWAY CANNOT PASS THE CURTIS WATERSHED PMF OR $\frac{1}{2}$ PMF WITHOUT OVERTOPPING. ASSUME OVERTOPPING OF CURTIS DAM RESULTS IN FAILURE OF CURTIS DAM AND ESTIMATE CONSEQUENCES.

AN ATTEMPT HAS BEEN MADE BY BERTLE IN INSPECTION, MAINTENANCE AND REHABILITATION OF OLD DAMS, ASCE, pp. 328-336 TO PRESENT AN ESTIMATE OF PEAK FLOW FROM A DAM FAILURE BASED ON THE HEIGHT OF DAM AND ACTUAL DAM FAILURE EXPERIENCE. ASSUME THAT A FAILURE OF CURTIS DAM WOULD FOLLOW THIS TREND, AND THAT THE TOTAL STORAGE VOLUME BEHIND THE DAM WOULD BE RELEASED.

CURTIS DAM HEIGHT = 34 FEET (DAMS, RESERVOIRS AND NATURAL LAKES)

FROM FIGURE 1, p. 329, PEAK FLOW = 82,000 CFS



SUBJECT: ELMHURST DAM (35-18) FILE NO. 7613.11
HYDROLOGY AND HYDRAULICS ANALYSIS SHEET NO. 10 OF 12 SHEETS
FOR: USCE - BALTIMORE DISTRICT
COMPUTED BY: JMC DATE: 5/5/78 CHECKED BY: DAW DATE: 5/9

INSPECTION, MAINTENANCE, AND REHABILITATION OF OLD DAMS, ASCE

SELECTING SPILLWAY FLOODS 329

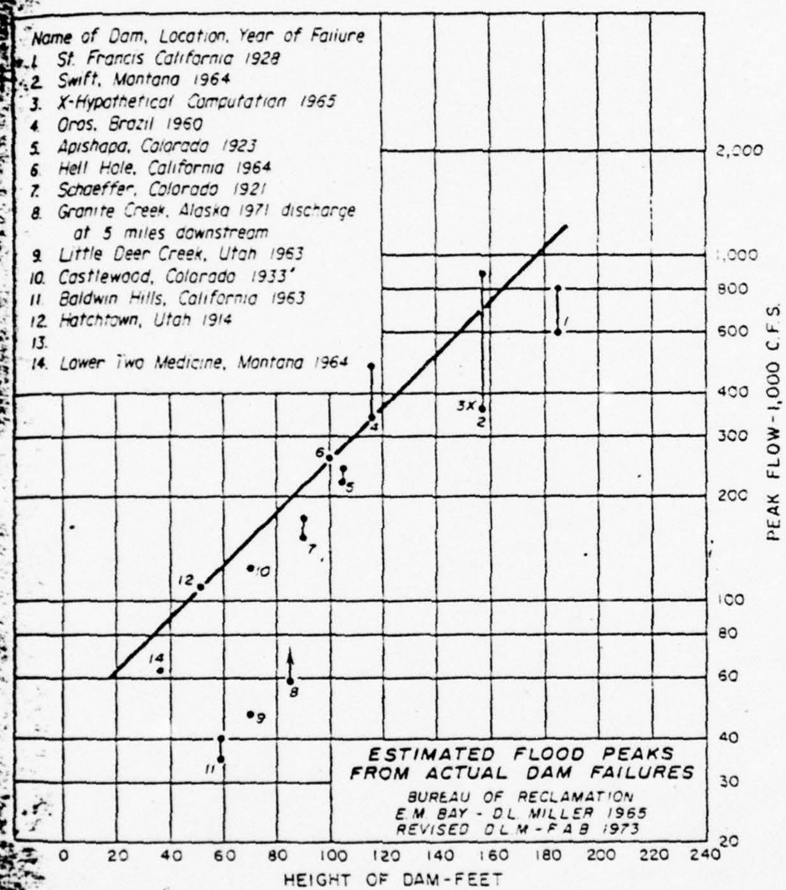


FIG. 1. -- ESTIMATED FLOOD PEAKS FROM ACTUAL DAM FAILURES

From:
"SELECTING SPILLWAY FLOODS FOR EXISTING STRUCTURES"
By Fredrick A. Bertle, 1973

GANNETT FLEMING CORDDRY AND CARPENTER, INC.
HARRISBURG, PA.

SUBJECT: ELMHURST DAM (35-13) FILE NO. 7613.1A
HYDROLOGY AND HYDRAULICS ANALYSIS SHEET NO. 11 OF 12 SHEETS
FOR: USCE - BALTIMORE DISTRICT
COMPUTED BY: JMC DATE: 5/2/79 CHECKED BY: DAW DATE: 5/5

STORAGE VOLUME = 418,333,000 GALLONS + 288 AC-FT

$= 418{,}333{,}000 \text{ GALLONS} \times \frac{1 \text{ FT}^3}{7.4805 \text{ GAL}} \times \frac{1 \text{ ACRE}}{43{,}560 \text{ FT}^2} + 288 \text{ AC-FT}$

$= 1{,}284 \text{ AC-FT} + 288 \text{ AC-FT} = 1{,}572 \text{ AC-FT}$

$= 1{,}572 \text{ AC-FT} \times \frac{43{,}560 \text{ FT}^2}{1 \text{ ACRE}} \times \frac{1 \text{ HOUR}}{3600 \text{ SEC}}$

$= 19{,}020 \text{ CFS-HOURS}$

$A = \frac{1}{2} b h$

$VOL = \frac{1}{2} \times TIME \times PEAK$

$1{,}9020 \text{ CFS-HOURS} = \frac{1}{2} \times T(\text{HOURS}) \times 82{,}000 \text{ CFS}$

$T(\text{HOURS}) = 0.464$

$T \approx 28 \text{ MINUTES}$

II. B. ABILITY OF ELMHURST SPILLWAY TO PASS RELEASE FROM CURTIS FAILURE

1. CAPACITY OF SPILLWAY = 31,000 CFS
3. INFLOW IS GREATER THAN SPILLWAY CAPACITY (82,000 > 31,000)
   b. ROUTING IS NOT AVAILABLE
      (1.) THE SPILLWAY WILL PASS (31,000 / 82,000) = 0.378 = 37.8% = p OF THE PEAK
      (2.) INCLOSURE 3 METHOD TO ESTIMATE STORAGE EFFECT OF RESERVOIR
         (a) TRIANGULAR SHAPE FOR FAILURE HYDROGRAPH
         (b) FAILURE HYDROGRAPH VOLUME = 19,020 CFS-HOURS = 1,572 AC-FT = ΔAOB

$1-p = 1.0 - 0.378 = 0.622 = \frac{\Delta AOC}{\Delta AOB}$

$\Delta AOC = (1-p)\ \Delta AOB = (0.622)(1{,}572) = 978 \text{ AC-FT} = \text{REQUIRED STORAGE}$

STORAGE REQUIRED = 978 AC-FT < STORAGE AVAILABLE = 1,646 AC-FT
IF ELMHURST RESERVOIR ELEVATION AT FAILURE IS AT SPILLWAY ELEVATION (1422.5') AND IF INFLOW TO ELMHURST RESERVOIR OTHER THAN INFLOW FROM THE CURTIS DAM FAILURE IS INSIGNIFICANT.

NOTE: THE PREVIOUS COMPUTATIONS DO NOT INCLUDE WAVE ANALYSIS OR SURGE COMPUTATIONS THAT MAY PRODUCE SPLASHOVER AT ELMHURST DAM.

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PERCENT OF PMF THAT SPILLWAY CAN PASS

GENERAL FORMULA

$$\%\text{ OF PMF THAT SPILLWAY CAN PASS} = \frac{Q_T}{Q_{PMF}} \times 100\% ,$$

$$\text{WHERE } Q_T = Q_{SPILLWAY} + \frac{2S}{\Delta T} ,$$

$$S = \sum_{i=1}^{n} S_i \quad \text{FOR UPSTREAM RESERVOIR CASES,}$$

AND T = TOTAL TIME OF PMF HYDROGRAPH FROM CURVE FOR SUSQUEHANNA RIVER BASIN

$$\%\text{ OF PMF} = \frac{31{,}000 + \left(\frac{2 \times (1{,}646 + 288)\text{ AC-FT}}{41.6\text{ HOURS}} \times \frac{43{,}560\text{ FT}^2\text{-HR}}{3{,}600\text{ AC-SEC}} \right)}{39{,}930} \times 100\%$$

$$= \frac{31{,}000 + 1{,}125}{39{,}930} \times 100\%$$

$$= 80\% \quad \text{FOR ELMHURST DAM}$$

$$\%\text{ OF PMF} = \frac{2{,}264 + \left(\frac{2 \times (288)\text{ AC-FT}}{22.0\text{ HRS}} \times \frac{43{,}560\text{ FT}^2\text{-HR}}{3{,}600\text{ AC-SEC}} \right)}{6{,}270} \times 100\%$$

$$= \frac{2{,}264 + 317}{6{,}270}$$

$$= 41\% \quad \text{FOR CURTIS DAM}$$

SUSQUEHANNA RIVER BASIN

ROARING BROOK, LACKAWANNA COUNTY

PENNSYLVANIA

ELMHURST DAM

PENNSYLVANIA GAS AND WATER COMPANY

PHASE I INSPECTION REPORT

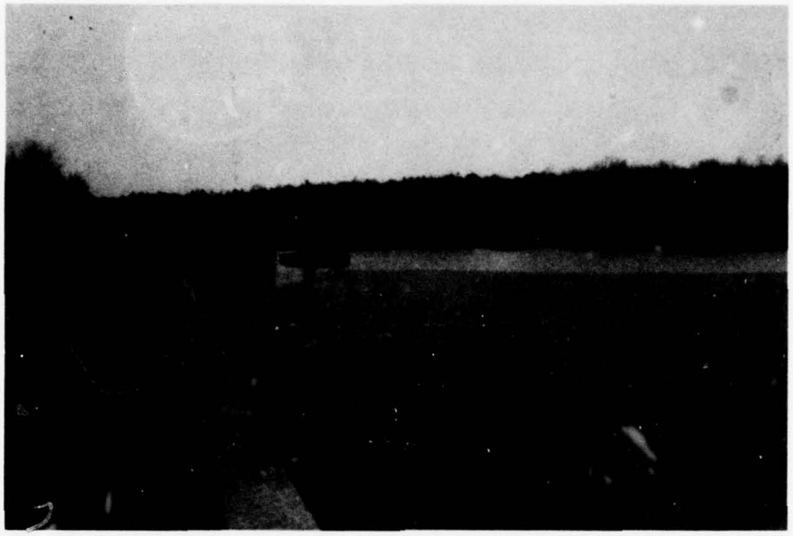
NATIONAL DAM INSPECTION PROGRAM

MAY 1978

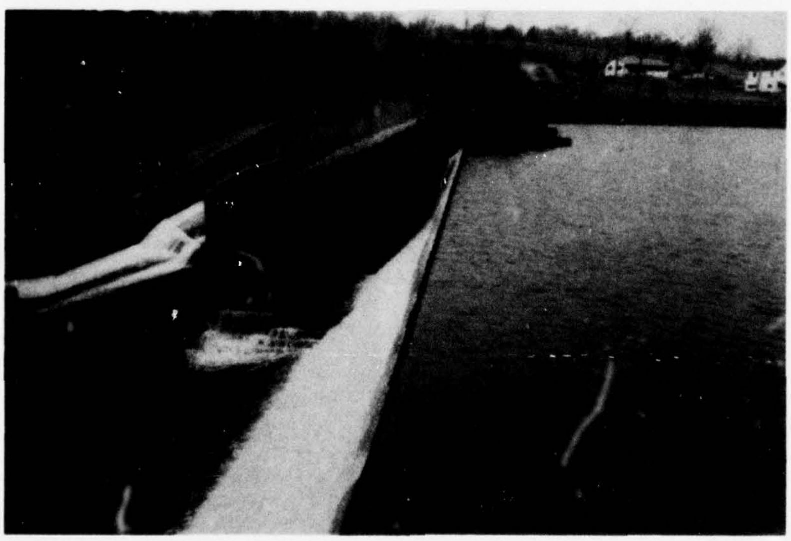
APPENDIX D

PHOTOGRAPHS

ELMHURST DAM

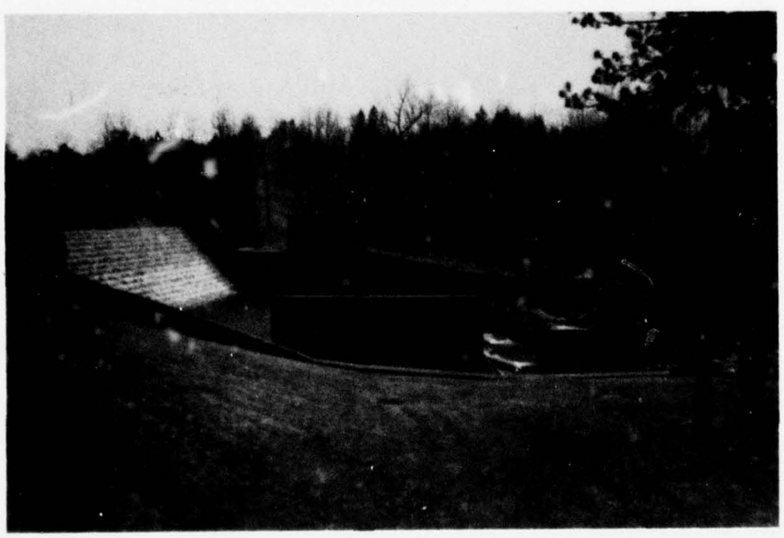


A. View from Left Abutment
Left Abutment Wall, Intake Structure,
Screen Chamber Building, and
Approach to Spillways



B. Masonry Gravity Spillway and
Concrete Chute Spillway

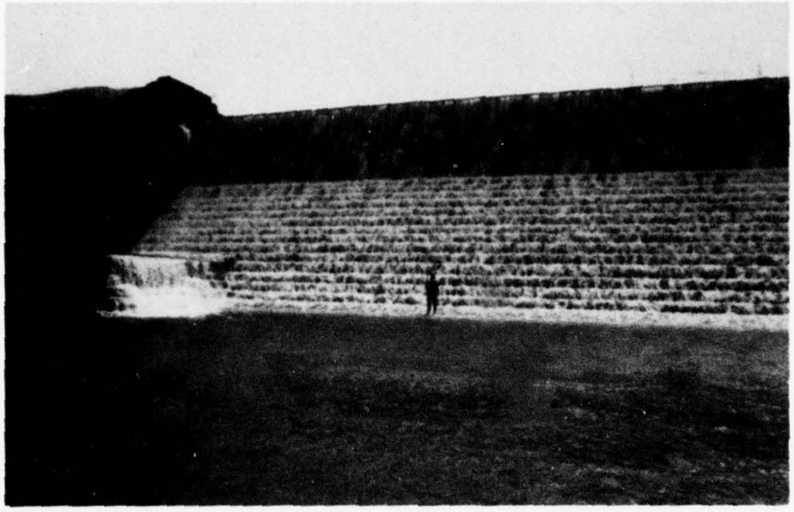
C. Concrete Chute Spillway



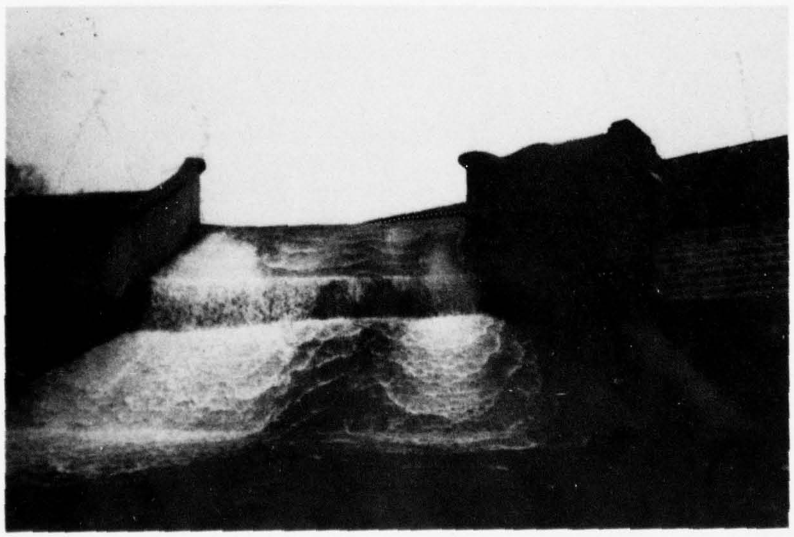
D. View from Downstream Right Abutment Showing Screen Chamber Building and Three Gatehouses

E. Area Adjacent to Embankment
Damaged by Surface Runoff

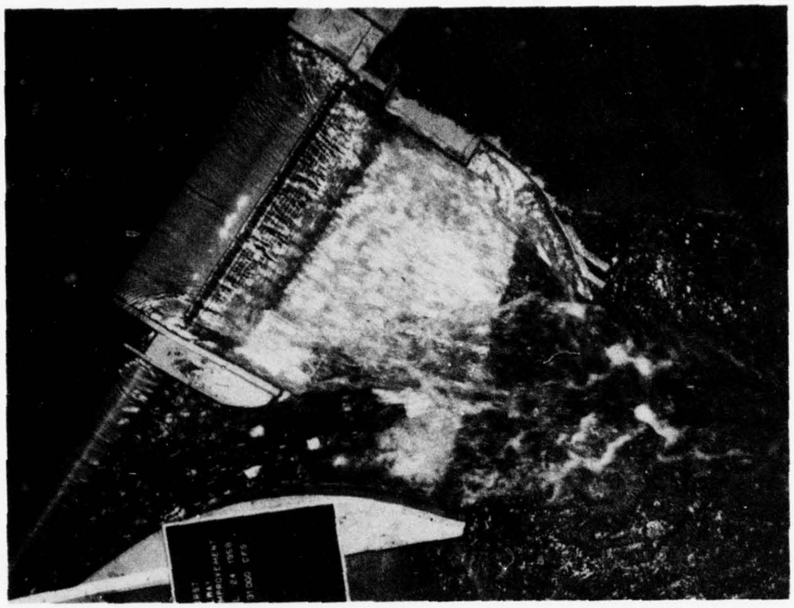
F. Concrete Apron of Masonry Gravity Spillway
and Downstream Channel



G. Masonry Gravity Spillway and Concrete Apron

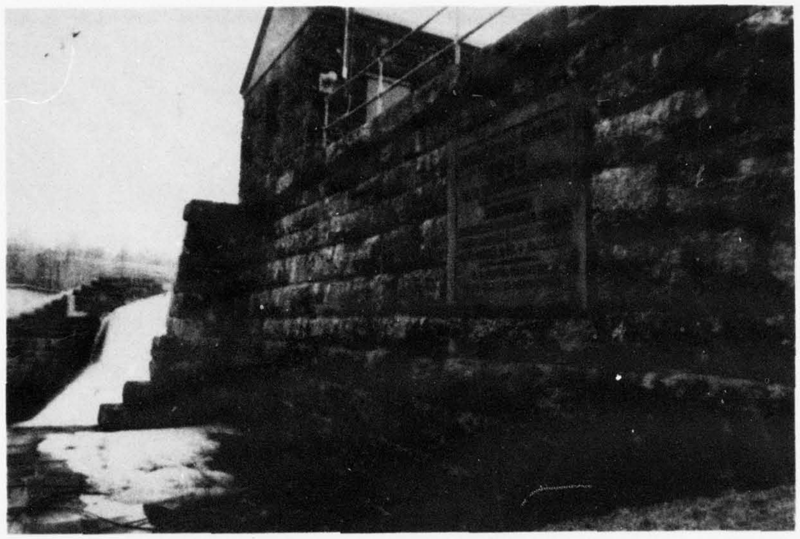


H. Concrete Chute Spillway and Apron



I. Discharge of 31,000 cfs on Model at Alden Hydraulic Laboratory (1958)

J. Discharge of 31,000 cfs on Model at Alden Hydraulic Laboratory (1958)



K. Downstream Face of Masonry Gravity Section and Screen Chamber Building

L. Right Wall of Concrete Chute Spillway at Apron

AD-A068 853 GANNETT FLEMING CORDDRY AND CARPENTER INC HARRISBURG PA F/G 13/2
NATIONAL DAM INSPECTION PROGRAM. ELMHURST DAM (NDS ID 296), SUS--ETC(U)
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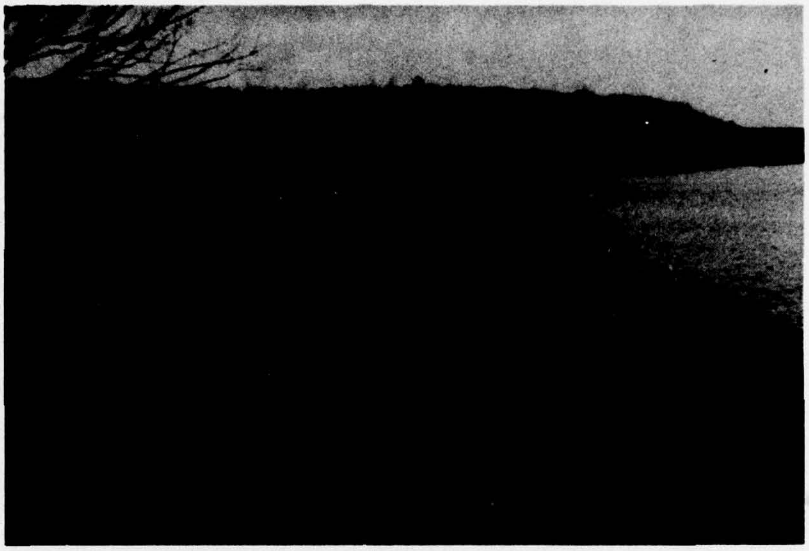
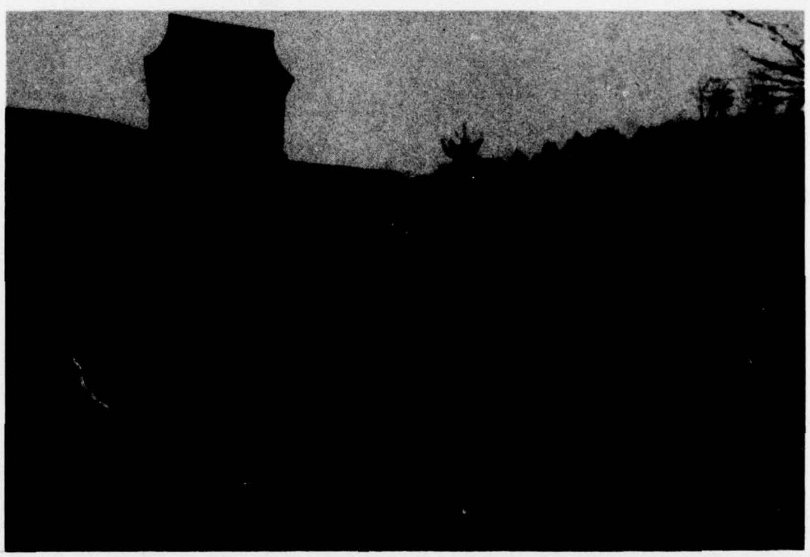
ELMHURST DAM



M. Curtis Dam
(Located 0.5 Mile Upstream from
Elmhurst Dam on White Oak Run)



N. Curtis Dam Spillway

SUSQUEHANNA RIVER BASIN

ROARING BROOK, LACKAWANNA COUNTY

PENNSYLVANIA

ELMHURST DAM

PENNSYLVANIA GAS AND WATER COMPANY

PHASE I INSPECTION REPORT

NATIONAL DAM INSPECTION PROGRAM

MAY 1978

APPENDIX E

GEOLOGY

ELMHURST DAM

APPENDIX E

GEOLOGY

1. General Geology. The damsite and reservoir are located in the northeastern portion of Lackawanna County. Lackawanna County was completely covered with ice during the last continental glaciation of Pleistocene time. The general direction of ice movement was S 35°-40° W. Glacial drift covers the entire County, except where subsequent erosion has removed it. Thick deposits of glacial outwash occur in many places along the Lackawanna River, and are 50 to 100 feet thick near Dickson, Scranton, and Moosic.

The only important structural feature in Lackawanna County is the Lackawanna Syncline, which traverses the County in a southwesterly direction. The syncline enters the County at the northeast corner as a narrow shallow trough, gradually deepens and broadens toward the southwest, and reaches its maximum development in Luzerne County. The rock formations exposed range from the post-Pottsville formations (youngest) through the Pottsville, Mauch Chunk shale, Pocono sandstone to the Damascus formation of the Catskill group (oldest). The rim rocks, the Pottsville formation and Pocono sandstone, have dips that rarely exceed 10° to 20° and form a rather simple syncline. The core rocks, the post-Pottsville formations, are folded into a series of minor anticlines and synclines which trend about N 70° E. The rocks in the northwestern and southeastern parts of the County, outside of the limits of the Lackawanna Syncline, are generally horizontally stratified.

The Lackawanna River, in general, follows the axis of the Lackawanna Syncline. Southeast of the Lackawanna River, the rise in terrain is quite gradual and the crests of the high mountains are several miles from the Lackawanna River. Streams, such as Roaring Brook and Stafford Meadow Brook, have cut deep canyons through the mountains and follow a tortuous course to their confluence with the Lackawanna River near Scranton, Pennsylvania. In the area of interest, the Lackawanna River streambed is founded in post-Pottsville formations. Proceeding uphill from the river, the older Pottsville formation, Mauch Chunk shale, Pocono sandstone, and Catskill continental group are encountered in turn. The tributary streams, in flowing down the mountains, have generally cut through or around the hard sandstone and conglomerate members, and have eroded their streambed into the softer shales and glacial till. The Catskill continental group of rocks underlies the greater part of Lackawanna County.

2. Site Geology. Elmhurst Dam is founded in the sandstones and shales of the Catskill group. The left abutment of the dam is keyed into an outcrop of hard, horizontally stratified, massive, red-and-green Catskill sandstone. As is characteristic of numerous other streams in this section of the State, the stream itself and the opposite, or right abutment, is founded in decomposed Catskill shale, probably the Damascus formation, or on glacial till. The 1914 reports of the Pennsylvania Water Supply Commission indicated that the left half of the dam was founded on rock; while the right half was founded on a stiff clay — with the abrupt transition occurring near the center of the masonry spillway. The rock, however, afforded a good foundation for the towers, and gate chamber and a runway for the blowoff pipe. The new spillway was constructed upon the original right-half embankment of the dam.